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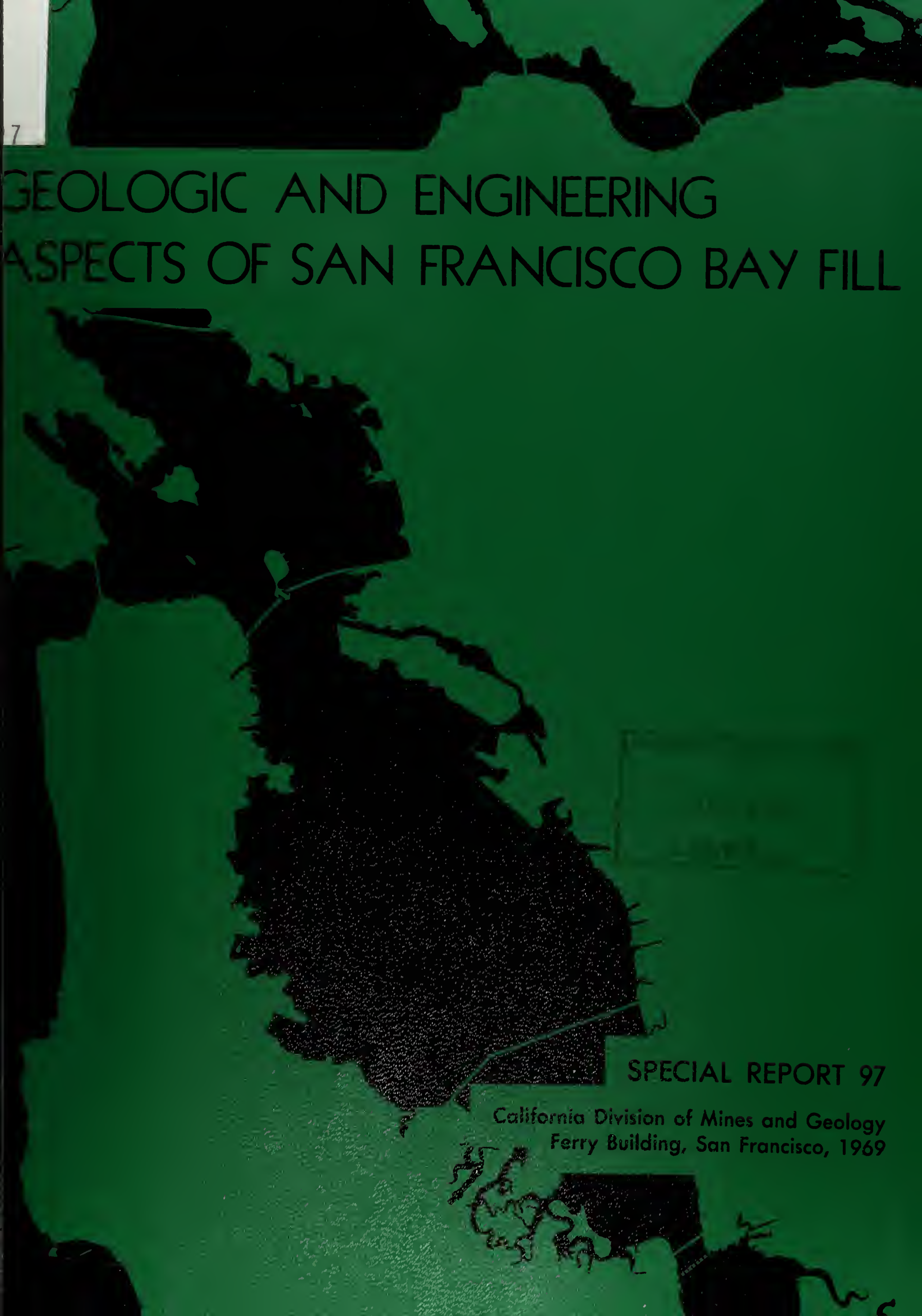
GEOLOGIC AND ENGINEERING ASPECTS OF SAN FRANCISCO BAY FILL



SPECIAL REPORT 97

California Division of Mines and Geology
Ferry Building, San Francisco, 1969

GEOLOGIC AND ENGINEERING ASPECTS OF SAN FRANCISCO BAY FILL

Prepared for San Francisco Bay Conservation and Development Commission

Harold B. Goldman, Editor

SPECIAL REPORT 97

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PREFACE

In 1965, the San Francisco Bay Conservation and Development Commission, or BCDC as it is popularly known, came into existence under Senate Bill 309. The commission was created in response to public concern with the piecemeal filling of San Francisco Bay.

Under the bill, the commission is directed to make a study of the bay to serve as the basis for preparing a comprehensive and enforceable plan for the conservation of the waters of San Francisco Bay and the development of its shoreline. As part of the study program, reports will be prepared on 23 separate aspects of the bay, such as water pollution, fish and wildlife, flood control, and tidal movements. Some study reports are prepared by BCDC staff; the remainder are prepared under contractual arrangement with other state agencies, or private consultants.

One area of great concern is the development of construction sites on marginal lands by reclamation or by filling. The relative stability of these sites depends not only upon the manner in which the site is prepared, but on the natural conditions that relate to the geology and the seismicity of the area.

Consequently, BCDC called upon the services of experts in the various disciplines: Prof. Harry Seed of the University of California for a discussion of seismic problems in the use of fills in San Francisco Bay; Lee and Praszker, Consulting Engineers, for a discussion of bay mud developments and related structural foundations; and Karl Steinbrugge, Structural Engineer, for a discussion of the seismic risk to buildings and structures on filled lands. Harold B. Goldman of the Division of Mines and Geology provided the technical background data on the geology and mineral resources within the bay.

These reports on geology, mineral deposits, earthquake hazards, and fill problems represent the latest data available and most recent thinking on these subjects. The Division of Mines and Geology is reprinting these reports to effect a wider distribution of invaluable data since the original printing by BCDC was quite limited and not widely distributed.

The reports are presented essentially as prepared for the BCDC except for some minor editorial changes. The plates for the report on geology have been combined into three separate maps.

ABSTRACT

San Francisco Bay came into existence in mid- to late Pleistocene time. The geologic formations underlying the bay fall into two distinct units: a bedrock composed of sandstone, siltstone, chert, and greenstone of the Franciscan Formation; and a younger unconsolidated sedimentary sequence which has been divided into older bay mud, sand deposits, and younger bay mud.

Seismically active faults bound San Francisco Bay which can be expected to experience strong earthquakes as a result. The soft, water-bearing sediments, such as the bay mud, may be expected to react strongly to such earthquake-generated shocks.

The mineral resources of San Francisco Bay are the sand deposits which occur in or adjacent to existing channels; shells that are imbedded in the bay mud in the southern end of the bay; and sea water as a source of salines. Ponds for harvesting salt cover the marshes for many miles along the shores of the South Bay and San Pablo Bay. In 1967, the value of these minerals produced exceeded 12 million dollars.

Engineering problems associated with filling the bay as urban expansion grows are related to the poor foundation characteristics of the bay muds. Contour maps based upon sparse subsurface data show variability in mud thicknesses and in the slope and configuration of the bedrock.

Almost all fills in San Francisco Bay are placed on top of the younger bay mud which is a compressible silty clay. Normal settlement of the fill takes place as the underlying mud is compressed. Case histories of fills at Treasure Island, the San Francisco waterfront, the U.S. Naval Air Station at Alameda, and City of Oakland Municipal Auditorium indicate that settlement rates can be calculated.

Factors affecting normal settlement are thickness of mud, method and rate of filling, slope of bedrock, and type of fill. A well-designed fill will prevent mud from squeezing out from underneath the fill.

Under earthquake conditions, the fill or the underlying mud or sand may fail when on a steep slope, may settle, liquefy, slide, or slump. Proper analysis and design is required to insure stable fills.

Buildings on fills must be designed to take into account stress induced by differential settlement. For houses on fill over bay mud spread footing is usually satisfactory; for one- or two-story industrial buildings, a series of foundations tied together under the entire floor can be adequate; heavier buildings should be on piles.

The potential for damage to buildings during earthquakes because of ground failure can be lessened by adequate soils engineering as the fill is placed, recognizing that structures located on loose, water-saturated material suffer far greater damage than those located on solid rock. A classification of risk zones is proposed to prescribe maximum safety in the development of potential bay fill areas.

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GEOLOGY OF SAN FRANCISCO BAY

By **HAROLD B. GOLDMAN**, Geologist
California Division of Mines and Geology

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GEOLOGY OF SAN FRANCISCO BAY

By Harold B. Goldman

INTRODUCTION

Purpose. The San Francisco Bay Conservation and Development Commission has been charged with preparing a comprehensive plan for the conservation of the waters of the bay and the development of its shoreline. An integral part of the plan will be the relationship between geology and urban development. The purpose of this report is to provide the Bay Conservation and Development Commission with general information on the geology of the bay, to help the Commission establish guidelines for future development in the bay.

Scope. The aspects of geology pertinent to this report are the types and distribution of sediments underlying the bay, and their suitability for filling and construction; the nature and configuration of the subsurface bedrock, particularly on the margins of the bay; the areas of filled lands and the underlying sediments; and the location of the major faults. The relationship of the types of sediments and their response to severe seismic activity is discussed by Professor H. Bolton Seed in an accompanying report.

A total of three months, from May to August, 1966, was spent in collecting, evaluating and preparing this report. The published as well as available unpublished literature was searched; data were compiled, synthesized, and reevaluated. Wherever possible, attempts were made to correct obvious errors in the published data and to add new information made available in the last few years.

James Kahle, geologist on temporary assignment with the Division of Mines and Geology for this interval, assisted the author in compilation of the data and preparation of the illustrations. Dr. Gordon B. Oakeshott wrote the section on geologic features of earthquakes of the bay area.

Acknowledgments. We wish to thank the U. S. Army Corps of Engineers for its excellent cooperation in providing the basic data and illustrations that were used as a prime source for this report. Geologists of the U. S. Army Corps of Engineers and the U. S. Geological Survey offered guidance in obtaining and evaluating new data made available since the Corps of Engineers' study for a salt water barrier was published in 1963. The consulting firms of Dames and Moore and Woodward-Clyde-Sherard and Associates kindly provided results of their borings on depths to bedrock and thickness of soft bay mud. Soils maps classifying the marginal lands on the bay were furnished by the U. S. Soil Conservation Service, through the cooperation of the Association of Bay Area Governments. The California Division of Bay Toll Crossings supplied new subsurface sections along proposed crossings in the South Bay and made available the results of their borings for other crossings. The Vallejo District office of the California Department of Water Resources was an excellent source of references about the bay. The Marine Technology Center at Tiburon of the U.S. Bureau of Mines allowed the writer to study specimens of the bay sediments recovered by various sampling techniques.

REGIONAL GEOLOGY

The regional geology provides a background for understanding the geologic events that led to the present configuration of the bay. The spatial relationship of the major active faults in the area is of direct concern because of the related seismic activity.

The geology of the San Francisco Bay area was mapped by Lawson (1914) at a scale of roughly one mile to the inch. Since then, the geology of several 7½ minute quadrangles has been mapped by the U. S. Geological Survey at a scale of 1:24,000 (see Bonilla; and Radbruch; and Schlocker in the bibliography). A series of quadrangle maps showing the geology north of San Francisco Bay was published by Weaver (1949).

The general geology of San Francisco Bay was discussed in some detail in the Geologic Guidebook of the San Francisco Bay Counties, Bulletin 154 of the Division of Mines (Jenkins, 1951). The articles therein formed the basis for the following discussion.

Geomorphology: Land Form

The San Francisco Bay area is a wide, north-trending depression that lies between crudely parallel ridges and extends north toward Santa Rosa and south through the Santa Clara Valley. On the west are ridges that comprise the San Francisco and Marin Peninsulas. They are separated by the Golden Gate channel which is the drowned valley of the ancestral Sacramento River.

On the east, the Berkeley Hills extend for 40 miles and are flanked on the west by a low, gently sloping plain that borders the east shore of the bay. The Berkeley Hills are cut off by the Sacramento River on the north.

San Francisco Bay is about 55 miles long and from three to twelve miles wide. Constrictions divide the bay into Suisun and San Pablo Bays on the north, and into north and south San Francisco Bays. Most of the bay is shallow with water depths of less than 10 feet except deep channels that represent drowned drainage systems are present. The deepest point, approximately 340 feet below sea level, is in the main channel through the Golden Gate. Islands such as Angel, Yerba Buena, and Alcatraz are the tops of buried hills.

One of the most striking topographic features is the San Andreas rift zone, which bisects Marin and San Mateo Counties, passing west of the city of San Francisco. The fault separates the Marin Peninsula on the east from the Point Reyes Peninsula on the west; and the San Francisco "block" from the Montara Mountain "block." The San Francisco-Marin "eastern block" includes the area east of the San Andreas fault zone and extends easterly to the Hayward fault zone near the western base of the Berkeley Hills. This block consists of a series of northwest-trending ridges of which San Bruno Mountain and Burri Burri Ridge are the principal summits, and the San Francisco Bay depression. Merced Valley lies between these two ridges and opens to the southeast into San Francisco Bay.

Stratigraphy: Rock Sequence

The San Francisco Bay area contains most of the rock units present in the central Coast Ranges. These units have been faulted and folded into structures typical of the Coast Ranges as a whole. The general distribution of these rocks, which range in age from Jurassic to Recent, is presented on figure 1.

Jurassic-Cretaceous (KJf-KJfv)

The oldest rock exposed in the San Francisco-Marin block are those of the Franciscan Formation, probably Cretaceous in this block. The San Francisco Peninsula is the type locality for the Franciscan, which was named by Lawson in 1895. This formation consists principally of interbedded sandstone and shale, limestone, radiolarian chert, and metavolcanic rock. The sedimentary rocks have been intruded by diabase, gabbro, and serpentine. Locally, metamorphism (change by heat and pressure) has produced glaucophane schists. Neither the top or the bottom of the Franciscan Formation has been identified, so the total thickness is unknown; however, it is at least 10,000 feet thick and may be as much as 40–50,000 feet thick.

Franciscan rocks are well-exposed in the city of Sa Francisco, east of the San Andreas fault in Marin an San Mateo Counties, in and west of the Haywar Chabot fault zone in the western margin of the Berke ley Hills; and comprise the bedrock underlying th alluvium and waters of the bay and exposures in th islands of the bay.

Table 1. The geological time scale, adapted from Mason, 1966.

Eras	Approximate age (in millions of years)	Subdivisions	Approximat duration (in millions of years)
Quaternary	.01	Recent	.01
	3	Pleistocene	3
Tertiary	12	Pliocene	9
	25	Miocene	13
	40	Oligocene	15
	60	Eocene	20
	70	Paleocene	10
Mesozoic	135	Cretaceous	65
	180	Jurassic	45
	225	Triassic	45
Paleozoic	270	Permian	45
	350	Carboniferous	80
	400	Devonian	50
	440	Silurian	40
	500	Ordovician	60
	600	Cambrian	100
Precambrian		Worldwide subdivisions not well established	2800 +

Oldest rock dated 3.5 billion
Age of the earth 4.5 billion

Basement Complex

The oldest rocks known in the bay area are sch and limestone that have been intruded by granite (m symbol gr). Exposures of these rocks, which are li ited to Montara Mountain, the southern end of t Point Reyes Peninsula, and the Farallon Islands, not occur east of the San Andreas fault. The age the granite has been placed as Late Cretaceous radioactive age dating.

Knoxville Formation (JK)

The Knoxville Formation of Late Jurassic age c sists largely of thin-bedded shale and some sandsto Small remnants of the formation crop out on t vest side of the Berkeley Hills and on the northwest ide of Suisun Bay south of Interstate 80.

retaceous (K, Ku, Kl)

Cretaceous marine sedimentary rocks crop out in cattered localities in the bay area. The most common ock types are intensely folded and faulted beds of lternating sandstone and shale that occur east of the Hayward fault in the Berkeley Hills, and to the north cross the Carquinez Straits, east of the Franklin fault one; and southeast of Mount Diablo. Locally, conglomerate beds are present near Black Point in Marin County and Niles in Alameda County.

aleocene (Ep)

Marine Paleocene shale and siltstone of the Mar- nez Formation unconformably overlie Upper Cre- ceous beds in a belt on the northeast flank of Mon- ra Mountain. Martinez massive sandstone is also ex- osed in the vicinity of Martinez and Benicia and on e north side of Mount Diablo.

ocene (E)

Marine Eocene sandstone and shale crop out in a lt west of Redwood City east of the San Andreas ult; near Martinez; and between Cordelia and Napa nction.

igocene

Oligocene sandstone and tuff occurs near Walnut eek in Contra Costa County.

ocene (M)

The lower Miocene is represented by coarse sand- one and minor conglomerate and shale of the Va- eros Formation which are exposed in San Mateo and nta Clara Counties. Middle and upper Miocene rocks clude the Monterey Formation, which is character- d by thin-bedded siliceous and diatomaceous shale, ert, and some sandstone and volcanic units. The onterey Formation and its equivalent are well ex- sed in the Berkeley Hills. Sandstone, tuff, shale, and nglomerate of the San Pablo Group occur in a wide t extending from San Pablo Bay to the southwest e of Mount Diablo.

ocene (Pml, Pmlc, Pc, Pv, Pu)

olded and faulted marine sandstone, mudstone, and le beds of the Purisima Formation are widespread southern San Mateo County. The Purisima, which ges in age from late Miocene to middle Pliocene, ins a maximum thickness of 10,000 feet. Strata of age in the Berkeley Hills are non-marine sedimen- and volcanic units such as the Orinda Formation the Moraga Formation.

ore than 5,000 feet of marine sand, silt, gravel, dy mudstone, and volcanic ash accumulated in late cene-early Pleistocene time to consolidate later he Merced Formation. This unit occurs near Lake rced south of San Francisco, and covers more than square miles in northern Marin County. It appears on the San Francisco and Marin Peninsulas. Possible correlatives in the east bay are thin, non-marine beds. A complex series of lava flows and tuff beds with associated water-laid sediments comprise the Sonoma Volcanics which overlies the Franciscan Formation, and interfingers with the Merced Formation in Marin and Sonoma Counties.

Upper Pliocene-Pleistocene (QP)

Poorly-sorted, lenticular gravel, sand, silt, and clay occur as alluvial fan deposits comprising the Santa Clara Formation east of the San Andreas fault zone in San Mateo and Marin Counties.

Mid-Pleistocene to Recent (Qm, Qc, Qt, Qb, Qal, Qs)

Poorly-sorted lenticular alluvial sand, silt, gravel, and clay comprise a number of geologic units that range from Pleistocene to Recent in age. Unconsolidated sand occurs in the Colma Formation on Angel Island and on the San Francisco Peninsula. Dune sand covers more than half of the northern part of the San Francisco Peninsula. The Alameda Formation contains sand, silt, clay, and gravel and underlies the bay and coastal plain alluvium on the east side of the bay, attaining a maximum known thickness of 1,050 feet.

Layers of clayey gravel, sand, silt, clay, and gravel of the Temescal Formation, Merritt Sand, and San Antonio Formation overlie the Alameda Formation on the plain between the Berkeley Hills and the bay, and underlie parts of San Francisco Bay.

Fine sand, silty clay, and mud of Recent age underlie the waters of the bay and some artificially filled areas on the margins.

GEOLOGIC STRUCTURE

The San Francisco Bay area lies within the Coast Ranges geomorphic province which is characterized by a series of nearly parallel mountain ranges that trend obliquely to the coast in a northwesterly direction. The alignment of major fault zones, fold axes, and geological units emphasizes this northwestern structural grain. This grain was developed in response to northeast-southwest compressional forces of unknown origin. The bay itself has a more northerly trend than the major structures which are therefore intersected at their margins.

The San Francisco Bay trough was formed by a combination of warping and faulting (bending and breaking of layers of rock) and is underlain by a down-dropped or tilted block.

The major active faults in the area are the San Andreas fault which transects the San Francisco and Marin Peninsulas, the Hayward fault at the base of the Berkeley Hills, and the Sunol-Calaveras fault at the west end of the Livermore Valley. The latter merges northward into the Franklin fault zone which is possibly inactive.

STRUCTURAL HISTORY

The geologic history of the bay area is characterized by a long record of extensive earth movement and seismic activity that is complicated by changes in sea level related to Pleistocene glaciations (within the last one million years).

The San Francisco Bay area as it is known today came into existence in mid- to late-Pleistocene time. Rocks of the Franciscan Formation were deposited in extremely deep, narrow troughs at the foot of the continental slope from late Jurassic to mid-Cretaceous time. Sediments of the Knoxville and Cretaceous formations were deposited on shallower marine shelves, as were also sedimentary and volcanic materials of the Tertiary period. After deposition of extensive Cretaceous and Tertiary sediments, the formations were folded (bent) and faulted (broken) by a series of earth movements into the northwest-trending structural pattern of the central Coast Ranges. During latest Miocene and much of Pliocene time, the bay block was high-standing and was being eroded to supply sediments both to its west and east.

The San Francisco Bay trough did not come into existence until the end of the Pliocene epoch or during the Pleistocene. It was formed by a combination of warping and faulting. The San Francisco-Marin block was tilted toward the east, with the western edge forming the San Francisco and Marin hills, and the depressed eastern edge forming the uneven depression in which the bay now lies. In Pleistocene time (within the last one million years) the trough was nearly filled with sediments, some of marine origin, others derived from the surrounding hills.

During the interglacial stages of the Pleistocene Epoch, the trough which became San Francisco Bay was flooded by the general rise in sea level due to the release of melt water from glaciers in other parts of the world. Bay mud was deposited during and after the melting of the continental glaciers in Wisconsin time.

GEOLOGIC FEATURES OF EARTHQUAKES IN THE BAY AREA

By Gordon B. Oakeshott

Alaska, California, and Nevada are the most seismic states in the United States; over 90 percent of the country's damaging earthquakes have originated in these three states. Abundant historical evidence exists (see the accompanying report by Karl V. Steinbrugge), to show that the San Francisco Bay area, in a section from San Francisco to San Juan Bautista, is one of the few areas in California in which there is currently a high incidence of earthquakes. The bay counties have experienced about 12 damaging earthquakes in the past century. From studies of repeated geodetic measurements across the major fault zones, seismicity, and calculated rate of strain accumulation, it seems reasonable to expect a great earthquake (comparable to the San Francisco earthquake of 1906) once in 60 to 100 years. The history of major and great earthquakes in the world indicates that there is no regular periodicity, so it is obvious that such figures cannot be used for earthquake prediction but only for indicated expected frequency. All evidence points to the conclusion that areas of historically high seismicit are the places where damaging earthquakes are mos likely to center in the future.

Figure 1 shows epicenters of earthquakes of magn tude 4 and greater in relation to the major faults. Bas data for the plot of epicenters come from the Un versity of California Seismograph Station, Berkele and have been compiled and published through 196 by the State Department of Water Resources (1964

Damaging earthquakes result from movements o faults that abruptly break the rocks of the outer po tion of the earth's crust along rough planes. In Cal fornia the depth of focus (point where the motio or earthquake, appears to originate) is usually on t order of 3 to 10 miles. Geologically, then, the reas for high frequency of earthquakes in the bay ar is apparent: there are major active faults on both sid of the bay. These are the San Andreas fault on t west and the Hayward and Calaveras faults on t east. These faults, and their branches, are close related and together comprise an important part of t San Andreas fault system. These three faults, ar probably some of their minor branches, are *acti* An active fault is one on which displacement has tak place in historic time as evidenced by linear patter of earthquake epicenters, or by surface slippage, "creep," along the fault trace. Some geologists al consider active those faults that mapping shows ha displaced geologically recent alluvium and along whi there are land forms, typical of fault zones, that ha not existed long enough to be greatly modified by t normal geologic processes of erosion and depositi Even if the San Andreas fault had not been the loc of many earthquakes and present-day creep in so places, we would recognize it as active (and therefo a potential source of earthquakes) because of its stri ing linear rift-valley features such as the long, narr Tomales Bay, the San Andreas and Crystal Sprin Lakes on the peninsula, sag ponds, notched ridges, a displaced and fault-oriented drainage features along t fault trace.

What Happens in an Earthquake

Recognizing, then, that the San Francisco Bay a is an area of active faults and high earthquake f quency, the critical consideration for development the bay in the best interests of public safety a welfare is, "What may reasonably be expected to ha pen in the event of a moderate-to-great earthquake The purpose of this chapter is to point out what happened, what does happen, and therefore what m be expected to happen in terms of the abrupt mo fication of ground (geologic) features. Results of cri cal field examination of the characteristics of hundre of faults, and particularly of the geologic featu accompanying dozens of earthquakes in western No America, supplies the best answers to such a questi Damage resulting from earthquakes is dependent up several specific geologic phenomena. Following is summary of the more important of these geolo features accompanying fault movements and ear quakes.

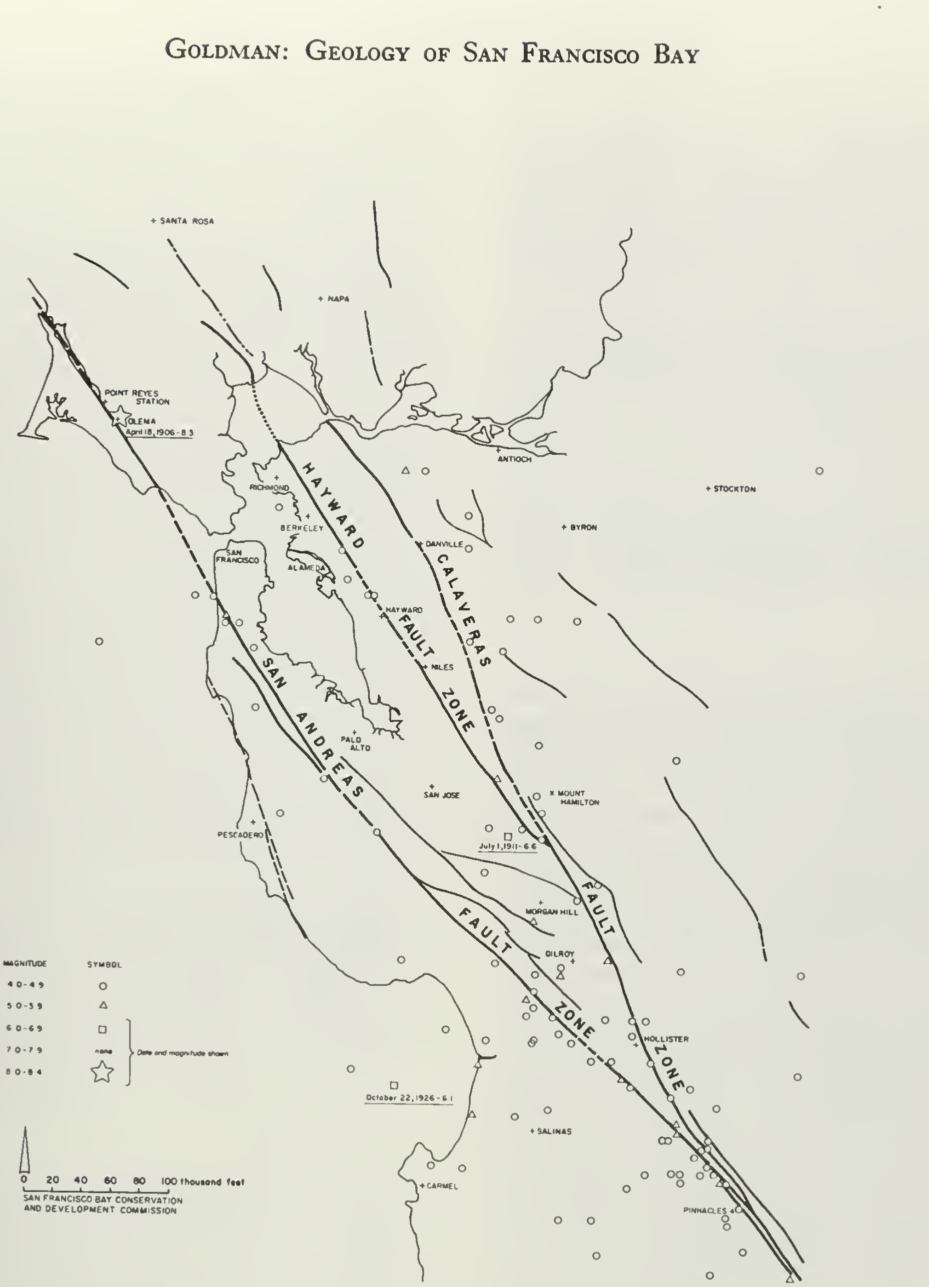


Figure 1. Earthquake epicenters and magnitudes, 1934–65. Source: Data from 1934–1962 from Department of Water Resources. After 1962 from other sources.

ound Shaking

Ground shaking—the earthquake itself—consists of
mplex surface wave motion which has traveled
ough the rock materials of the outer crust. Earth-
ake waves, like other waves, may be reflected, re-
cted, attenuated, and change velocity and period as
y pass through different materials, thus making the
ound motion complex. In general, earthquake waves,
passing from more dense solid rock to less dense
ivial and water-saturated materials, tend to become
uced in velocity, increased in amplitude, and accel-
tions become greater. Ground motion lasts longer
loose, water-saturated, incompetent materials than
rock. It is also amplified to an unknown extent.
e to a combination of factors, structures located on
such materials suffer far greater damage than those located on solid rock. This has been repeatedly and strikingly demonstrated in large earthquakes, including Tokyo 1923, Fukui 1948, Arvin-Tehachapi 1952, Chile 1960, and Alaska 1964. In all these cases, and many others, the violence of ground motion in soft alluvial materials was significantly greater than in solid rock. Amplification of ground motion is greatest in unconsolidated, saturated sediments.

To bring the experience closer to home, the Earthquake Commission Report on the San Francisco 1906 earthquake documented exaggerated shaking in the lower waterfront areas of San Francisco, underlain by the thickest bay mud and fill, as compared to lesser shaking in Nob Hill and similar areas with more solid rock at or near the surface. Many earthquakes have shown that "poor ground" is a greater hazard than proximity to the fault and epicenter. In 1906, Santa Rosa and San Jose, both underlain by alluvium, suffered to an extent out of proportion to their distances from the fault movement. There is some evidence to show that ground motion tends to increase with the depth of alluvium.

Unfortunately, comparatively little instrumental data exists on the difference in response of different types of rock and soil to wave motion. One of the greatest research needs is to establish a close network of strong-motion seismic instruments on the different kinds of rock and soil in many spots in and around the bay to monitor and record the effects of the dozens of minor —and the occasional major—earthquakes felt in this area every year. This would yield extremely valuable data fundamental to development of the bay and its margins.

Surface Faulting

Fault rupture, which begins at the focus, may extend to the surface, especially in moderate to large earthquakes. The direction of separation of the earth on opposite sides of a fault plane may be vertical, horizontal, or oblique. In moderately large earthquakes, surface scarps (the exposed scar) may well be as high as 20 feet (Hebgen Lake, Montana, 1959, magnitude 7.1) or, in smaller earthquakes, only a few inches high (Herlong, Plumas County, California, 1950, magnitude 5.75). Recently observed movements on the San Andreas, Hayward, and Calaveras faults seem to be of the horizontal type, east block moving relatively south. Maximum displacement of this type, San Francisco 1906, magnitude 8.25, was 20 feet, just south of Tomales Bay; Hayward 1868 was 3 feet; Fort Tejon, 1857, was perhaps as great as 30 feet. Arvin-Tehachapi 1952, magnitude 7.7, on the White Wolf fault, had oblique-slip displacement amounting to about 2 feet of upward movement on the south and a like amount of horizontal movement, south block moving eastward. There was no surface faulting in the San Francisco 1957, magnitude 5.3, earthquake. While historical displacements on the faults of the San Andreas system have been predominantly horizontal, there is much geological evidence to show that slightly older movements have involved a great deal of vertical displacement. Vertical movement on faults in future earthquakes in the bay area cannot be ruled out. Judging from geology and earthquake history, it appears unlikely that important active faults exist in rock under the bay itself, except that the Hayward fault probably extends to the north across San Pablo Bay. Surface faulting not only develops scarps, but also trenches (graben), patterns of ground fractures, "mole tracks" or pressure ridges, and open or closed fissures.

Recently, evidence has been accumulating that spasmodic slippage ("creep") is now occurring on the Hayward fault trace and on the San Andreas fault, with many individual slips not accompanied by felt earthquakes.

Faulting not only disturbs the ground surface but also disturbs and dislocates drainage features, both natural and artificial. Springs and ground-water flo characteristics may be changed.

Landsliding

Landslides, rock falls, avalanches, mud and deb flows and general gravitational movements of loc and weathered rock and soil are major effects ground shaking in earthquakes of magnitude 5 a greater. Such mass movements are readily triggered earthquakes, especially where slopes are oversteepen from whatever cause, and where earth materials a water-saturated, from artificial means or the natu rainy-season. An immense amount of sliding takes pla in large earthquakes but even small earthquakes, su as San Francisco 1957, cause many damaging slid At Hebgen Lake, Montana 1959, a rock slide carri 43,000,000 cubic yards of loose rock across the Ma son River. State Highway 1, on the coast south San Francisco, was closed by large landslides in t small earthquake of 1957. The San Andreas, Haywa and Calaveras fault zones are marked by a successi of many geologically-recent slides, many of th highly unstable. The steep hills on both sides of t bay are particularly vulnerable to sliding.

Recent work has shown that certain types of san and clayey sediments become "quick" as a result shaking and lose all cohesive strength. Such materi may flow like a thick fluid down extremely low-an slopes. Flow tends to be toward a "free" face, ev though the flow may be nearly horizontal. Liquef tion was a major factor in the disastrous slides in t Turnagain Heights area of Anchorage in the Alask earthquake of March 27, 1964. Whether such s liquefaction would take place in San Francisco B muds is not known; research is greatly needed on t properties and characteristics of bay mud. (See t accompanying report by H. Bolton Seed.)

Lurch Cracking

The development of all types and sizes of irregu fractures, cracks, and fissures—largely as the result sliding, settling and shaking, and the passage of surf earthquake waves—is characteristic, to a greater lesser extent, in all earthquakes large enough for s nificant ground motion to occur. Such fractures m be many feet long and may displace surface weathe rock and soil both horizontally and vertically, just true fault movement would. Lurch cracks rarely oc in solid rock; they are essentially confined to wea ered rock, alluvium, and soil. Their patterns may completely irregular or may show marked regular where controlled by shallow bedrock, by the outl of natural or artificial fill, by highway surfacing, e Lurch cracking is often accompanied by sand b and mud volcanoes as groundwater moves to the s face. Extensive and damaging lurch cracking has curred in incompetent water-saturated materials in moderate to large earthquakes (magnitudes 6 to Damage resulting from this type of ground fract was extensive in the waterfront areas of San Franci in 1906, in the irrigated alluvium and soil of the sou ern San Joaquin Valley in the Arvin-Tehachapi ear quake of 1952, in the Long Beach earthquake of 19 and in similar materials in every earthquake of damaging intensity. Undoubtedly, such cracking will be a major damage factor in many areas of bay mud and fill in future earthquakes of moderate to large magnitude in, or near, the bay. Lurch cracking may occur in water-saturated sediments, soils, and alluvium at distances up to 75 miles from the epicenter, as demonstrated most recently in the Alaskan earthquake. There is some evidence to indicate that lurch cracking is perhaps most extensive in the deeper alluvial materials.

Subsidence, Slumping, Compaction, Uplift

Especially in great earthquakes, such as San Francisco 1906 and Alaska 1964, but also in moderately large earthquakes, such as Hebgen Lake 1959 and Arvin-Tehachapi 1952, there are extensive changes in level and elevation of the land surface. These may take the form of broad uplift or subsidence, tilting of limited to large areas, and local slumping on the order of massive landsliding. Such earth movements may even pass unnoticed in inland areas, but become major factors along the shores of seas, bays, and lakes. Since historic displacements on the faults of the San Andreas system have been largely *horizontal*, this category of ground movements has not so far been of overriding local concern. However, even small upward, downward, or tilting movements become of great concern along a highly developed shoreline, such as that of the bay.

Compaction of soils and alluvial materials may take place extensively, and irregularly, in soils and fine-grained water-saturated sediments. In an earthquake, this takes place primarily as a result of shaking; compaction is induced by the vibration. It may amount to a few inches or several feet in soft materials; it does not occur in firm rock.

Seismic Sea Waves (Tsunamis) and Seiches

The seismic sea wave, tsunami, or "tidal wave" is probably caused by faulting and other abrupt ground movements on the ocean floor in connection with a major earthquake. Such waves move at high velocity, as much as 300 to 400 miles per hour in the open sea, may have a wave-length of many miles, and have low relative amplitudes. At sea, they are not noticeable but pile up water on shore to heights of up to perhaps [illegible]0 feet. Search of the historic records shows that no tsunami has raised sea level along the California coast more than a few feet. This coast, nevertheless, has been visited by such waves several times, generated by distant earthquakes. Crescent City, California, was damaged and several lives were lost by this type of wave (which arrived close to the time forecast in warnings issued by the U.S. Coast and Geodetic Survey) caused by the Alaskan earthquake of 1964; the California coast was also affected by a tsunami in connection with Chilean earthquake of 1960. A rise of water of even 2 or 3 feet in San Francisco Bay due to tsunami, if coupled with a high tide and on-shore wind, could do serious damage to near-sea-level developments. The tsunami warning system of the U. S. Coast and Geodetic Survey is efficient and should give local warning in ample time to reduce loss of life to almost zero. Unfortunately, experience has shown that people do not always heed such warnings.

A seiche is a periodic oscillation, or "sloshing" of water in an enclosed basin such as that of a lake or bay. It is usually due to earthquake motion, in connection with a moderate to large shock. The periods of oscillation range from minutes to hours, depending on the size and configuration of the basin affected. In great earthquakes, seiches have been observed several thousands miles from the epicenter. Seiches could only be damaging in San Francisco Bay in the event of a large earthquake and "right" combination of tide and wind.

Conclusions

San Francisco Bay is well known as an area of high earthquake frequency. In addition to many minor shocks, expected frequency of damaging earthquakes is about 12 per century; at least one of these each century may be expected to be a great earthquake, comparable to San Francisco 1906.

Geologic disturbances that accompany moderate-to-strong earthquakes in the bay area are: surface fault ruptures involving horizontal breakage up to 20 feet and possible vertical displacements of several feet; intense ground shaking; extensive landsliding and associated mass movements off unstable slopes; extensive lurch cracking confined to alluvial materials, natural and artificial fill; compaction and perhaps soil flowage on very low slopes in bay mud; relatively small seismic sea waves and seiches to the possible extent of causing shoreline damage, originating from distant as well as local earthquakes.

Geologic and seismologic considerations, supported by the historic record, show that earthquake shaking and accelerations, lurch cracking, slumping, and compaction are all greatly amplified in water-saturated or loose alluvial sediments, as compared to such features developed in adjacent solid rock.

Research, partly under way in the bay area, must be increased along these lines:

1. Detailed geologic mapping and soils surveys;
2. Determination of the properties and characteristics of bay muds and other natural materials on the bay margins;
3. Seismologic studies, including particularly strong-motion work in the response-to-shaking of bay muds and local rock formations;
4. Earthquake-resistant design and construction of buildings;
5. Code requirements concerning foundation materials as well as structures; and
6. Construction of geologic-hazards maps as a basis for zoning the bay and its margins.

GEOLOGY OF SAN FRANCISCO BAY PROPER

Previous Work

One of the pioneer works on the geology of San Francisco Bay and its influence on engineering problems was published by Trask and Rolston (1951), in conjunction with an investigation of two proposed crossings of the bay.

In 1955, the California Department of Water Resources released a report on the feasibility of construction by the State of barriers in the San Francisco Bay system. The geology appendix, H, contains data on the thickness and depth of the mud in the bay obtained from a compilation of more than 2,000 logs of borings.

Since then the work of the U. S. Geological Survey (Schlocker et al., 1958; Radbruch, 1957; Radbruch and Schlocker, 1958; Bonilla, 1965) has contributed much to the knowledge of the geologic units on the margins of the bay.

The study by the U. S. Army Corps of Engineers (1963), for a salt water barrier contains a wealth of data on the distribution and engineering properties of the sediments in the bay, and on configuration of the bedrock surface. Appendix E from their publication provided the basic data for the illustrations in this report.

Treasher (1963) discussed the characteristics of the sediments in the bay, the geologic history of the bay, and refined the contour maps published by the Corps of Engineers.

The distribution of the sediments on the floor of the bay was discussed in a paper by Trask (1953).

Geologic Formations

The geologic formations underlying San Francisco Bay are divided into two distinct units that differ greatly in age, type of rock, and mode of occurrence: an older unit that forms the bedrock, and a younger, unconsolidated sedimentary sequence.

Bedrock Unit

The bedrock underlying most of San Francisco Bay is composed of sandstone, siltstone, chert, and greenstone of the Franciscan Formation. Upper Cretaceous sandstone forms the bedrock in the Carquinez Straits. The bedrock crops out on the margins and on the surface of the bay and is encountered in drill holes. The distribution of the Franciscan is shown on the geologic map, plate 1.

The general configuration of the bedrock surface insofar as known is shown on the contour map, plate 2. The bedrock surface drops off sharply on the margin of the west side of the bay and deepens to the east. Based upon sparse information, bedrock is generally deepest in the southern part of the bay where depths of 300 to 800 feet are common. Radbruch (1957) lists depths near Alameda that range from 300 feet to 400 feet, with one drill hole there encountering "bedrock" at roughly 1,050 feet, reinforcing the concept of eastward tilting of a downthrown block. The bedrock surface, which was modified by stream erosion prior to deposition of the youngest sediments, and the location of ancient drainage divides is known only from drill holes and subbottom profiles. The bedrock surface is very irregular.

The problems of the bedrock relate primarily to the strength of the rock for foundation purposes and the slope of the bedrock surface. A surface of considerable relief and variation in depth of weathering was developed on the Franciscan rocks prior to deposition of the bay muds. Because the weathering weakened the rock surface, placement of foundations upon steeply sloping parts of the old topography should be thoroughly investigated so that proper foundation design can be provided to avoid problems.

Table 2. Correlation chart of sediments of San Francisco Bay.*

Lawson, A. C. (1914)	Trask, Parker, and Rolston, J. W. (1951)	Recent studies by Corps of Engineers	
Bay mud	Bay mud	Younger bay mud	Soft member
			Semi-consolidated member
Merritt Sand	Merritt Sand	Sand deposits (including Merritt Sand)	
San Antonio Formation	Posey Formation	Older bay mud	
	San Antonio Formation		
Alameda Formation	Alameda Formation		
Bedrock	Bedrock	Bedrock	
(Bedrock is principally Franciscan Formation)			

* Reprinted from Treasher, 1963, p. 15.

Younger Unconsolidated Sedimentary Sequence

Sediments accumulated to thicknesses of more than 300 feet during the formation of the present bay. This sedimentary sequence was subdivided by Trask and Rolston (1951) into five formations so as to correlate with the formations that Lawson had mapped on the east side of the bay. Subsequent work by the Corps of Engineers (1963) and Treasher (1963) question the validity of extrapolating these correlations for any distance from the area where Trask recognized these subsurface units. Treasher felt the units are so uniform in grain size and mineralogy that the principal differences were the degree of consolidation and stratigraphic position. In his study, he divided the sediments into an older bay mud, sand deposits, and a younger bay mud. The younger bay mud he further divided into a semiconsolidated member below and a soft member above.

Because of the lack of time to re-evaluate the thousands of individual drill logs and make new correlations and interpretations, it was decided to adopt for this report the units as used by the Corps of Engineers and Treasher. A correlation chart is presented in table 2. Geologic sections across northern San Francisco Bay are presented in figure 2. Sections across southern San Francisco Bay are presented in figure 3.

Older Bay Mud

The term "older bay mud" is used to describe a deposit of principally firm, dark greenish-gray, silty

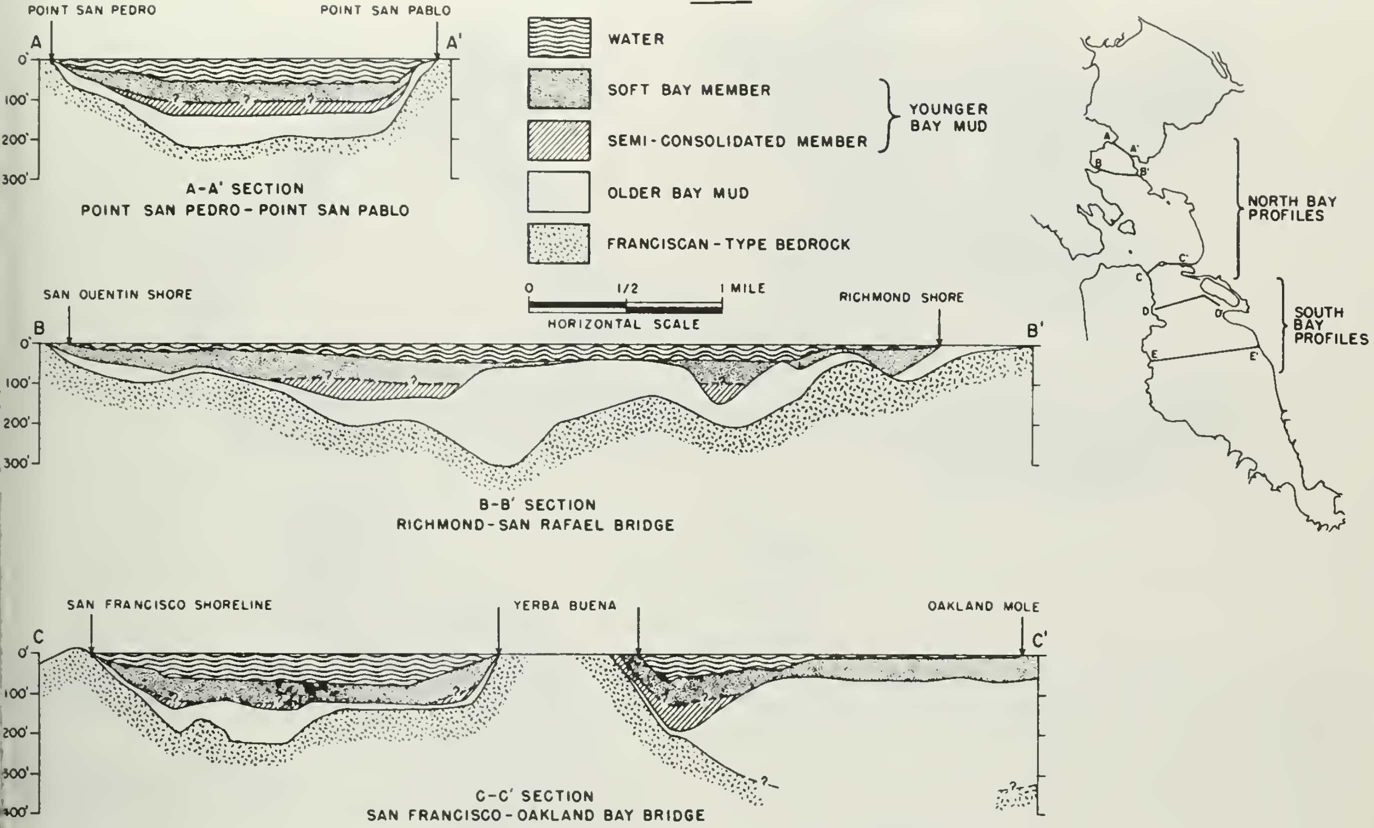


Figure 2. Profiles across northern San Francisco Bay, from Treasher, 1963.

clay, with varying amounts of sand and fine gravel that blankets and fills old stream channels cut in the bedrock, and which is overlain either by a sand unit or "younger bay mud." The older bay mud corresponds to the Alameda, San Antonio, and Posey Formations of Trask and Rolston (see table 2).

Parts of the older bay mud contain lenses of clayey sand, pebbly sand, and sandy clay from 5 to 50 feet thick. The thickness of the older bay mud unit ranges from less than a foot to more than 200 feet and appears to increase toward the central portion of the bay. Along the margins, the older bay mud unit may be missing.

In the Islais Creek area, the clay-size fraction (minus 2 microns) has been described by Radbruch and Schlocker (1958) as containing about 50 percent normal and hydrated mica with lesser amounts of montmorillonite, chlorite, diatoms, carbonaceous matter, quartz, feldspar, and kaolinite. The silt fraction contains quartz, feldspar, lignitic material, mica, and diatoms. Sediments of similar lithology are being deposited presently in shallow, relatively still waters where the tidal action is weak (Louderback, 1939, p.784).

The older bay mud differs in physical properties from the overlying units. Because the older bay mud is more deeply buried, it contains less moisture, and is overconsolidated (or preconsolidated); i.e., the degree of consolidation is greater than would be expected from the weight of the sediments that overlie the clay today. Preconsolidation may develop from the weight of overburden, drying upon exposure to air after deposition, or chemical action. Treasher (1963) suggests vibration due to earthquakes as another mechanism.

The older bay mud unit presumably was deposited during an interglacial period, prior to the advent of the Wisconsin Glaciation (late Pleistocene time), when glacial melt-water raised the sea level so as to flood the San Francisco Bay depression. In the next glacial stage (Wisconsin Glaciation), the sea level was lowered, and exposed the older bay mud and presumably resulted in preconsolidation.

Sand Deposits

A distinctive sand unit that overlies and interfingers with the older bay mud has been recognized in certain portions of the bay underneath younger bay mud. On the east side of San Francisco bay, a fine sand, 10 to 50 feet thick, overlies the older bay mud between Point Richmond and Bay Farm Island (Trask and Rolston, 1951). The sand is dissected by mud filled channels and is overlain by younger bay mud from 0 to 35 feet thick. A sand layer with a maximum thickness of 50 feet underlies much of the Islais Creek Basin (Radbruch and Schlocker, 1958). This sand unit has been correlated with the Merritt Sand of Lawson.

Sand is also present on the bay floor in a number of shoal areas such as Point Knox, Presidio, Southampton, and San Bruno. This sand grades laterally and vertically into sandy silt and sandy clay and may have a thin cover of mud over it. Logs of drilling in these

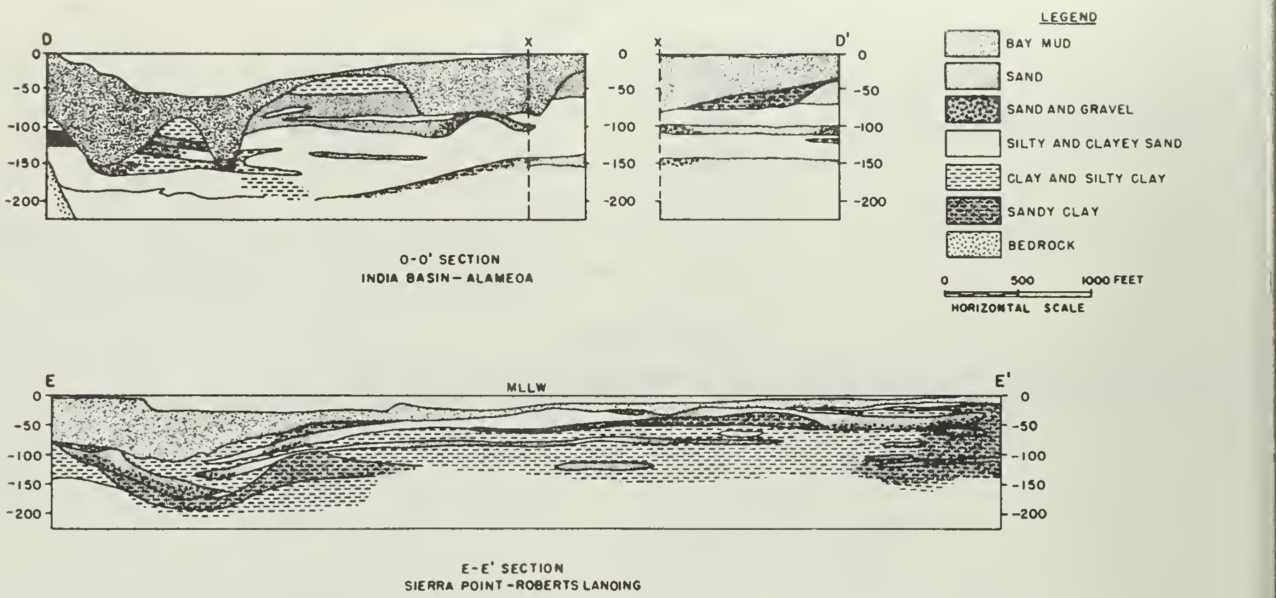


Figure 3. Profiles across southern San Francisco Bay, from California Division of Bay Toll Crossings.

deposits do not show a uniform thickness of sand, nor do all the holes show sand (U. S. Army Corps of Engineers, 1963, p.37). It is not known whether these sand deposits are correlative with the Merritt Sand, or are contemporaneous in age with the younger bay mud.

The known distribution of the sand deposits is so erratic that no attempt has been made in this study to draw contours on top of this unit.

Younger Bay Mud

Younger bay mud is the youngest unit in San Francisco Bay, overlying the older bay mud and (Merritt?) sand unit, and covering most of the bay bottom. This unit, which consists of a soft, uniform, gray, silty clay containing 45 to 95 percent clay-size particles, silt, minor fine sand, and fragments of shells, tends to become firmer and contain less water with increasing depth. The clay, which is similar to the clay of the older bay mud unit in mineral composition, contains mica, montmorillonite, chlorite, kaolinite, quartz, and feldspar. The clay is soft and plastic when wet, and tends to shrink, harden, and become brittle upon drying. On the basis of engineering properties, Treasher (1963) divides the younger bay mud into a lower semiconsolidated member and an upper soft member above.

The younger bay mud was deposited upon an erosional surface cut to depths of as much as 200 feet below sea level in the older bay mud, and attains thicknesses of as much as 130 feet. Contours on the bottom of the younger bay mud are presented on plate 3. The thickness of the younger bay mud is presented on plate 4 and figure 4.

The younger bay mud was formed during and after the melting of the Wisconsin continental glaciers. Treasher (1963) postulates at least three stages of flooding and two stages of draining the bay valley; with the first flooding to allow deposition of the older bay mud, the second and third floodings to permit deposition of two members of the younger bay mud—one now semiconsolidated.

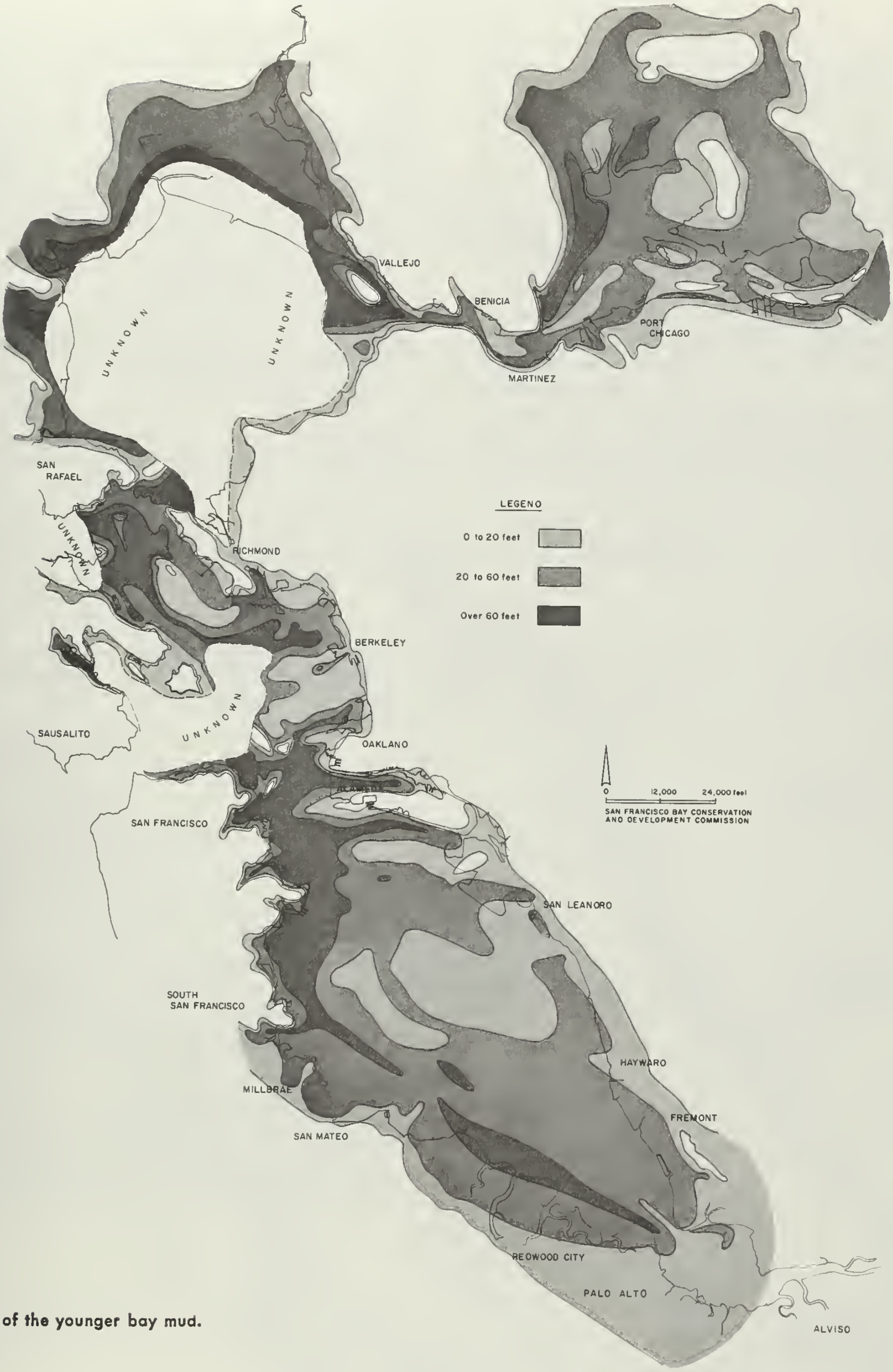
Radiocarbon dating from samples in the younger bay mud have yielded ages ranging from 7,360 years to 2,420 years old (Storey et al., 1966).

ENGINEERING PROPERTIES OF THE YOUNGER BAY MUD

Geologists and engineers have long recognized that from the soil mechanics and foundation engineering standpoints the younger bay mud is the most troublesome of the sediments of the bay. The foundation problems stem from the nature of the bay mud, which is primarily a soft, silty clay. The clay has a high natural water content, is quite plastic and weak, and highly compressible. These properties are, in a large measure, a reflection of the minerals in the contained clays which swell and become plastic in the presence of water.

In general, the unit weight and strength of the younger bay mud increases with depth as a result of the increase in pressure with depth. For example, in a 100-foot interval, the average water saturated weight will increase from 97 pounds per cubic foot to 110 pounds per cubic foot, while the compressive strength increases from 200 pounds per square foot to 1,000 pounds per square foot. These and other properties are described and compared with those of older bay mud in table 3. In this table, the younger bay mud has been subdivided into two members based solely upon the engineering properties: a soft member, and an older semiconsolidated member that is preconsolidated to a degree greater than would result from the weight of the overlying sediments. The amount of preconsolidation is considerably less than in the older bay mud.

In detail the engineering properties vary depending upon the thickness, depth, and the location of the mud.



gure 4. Thickness of the younger bay mud.

The main engineering problems, which arise from e low strength and high compressibility, are failure f the mud due to imposed load, and differential settleents (Mitchell, 1963). Differential settlement results rom variability in the thickness of the mud.

When younger bay mud is overloaded by fill, it beomes increasingly unstable as the height of the fill creases and, if the slopes on the edges of the fill e steep, failure ultimately results. During construction of the mole fill north of the Toll Plaza of the San Francisco-Oakland Bay Bridge in 1947, the mud was overloaded with a sand fill and failed. The sand sank 20 feet and the underlying mud was forced laterally for more than 500 feet.

The high water content, low strength, and high compressibility cause the mud to consolidate markedly under sustained loads. At the mole fill mentioned above, settlements of more than 8 feet took place

*Table 3. Engineering properties of sediments of San Francisco Bay.**

Properties	Older bay mud	Younger bay mud	
		Semi-consolidated member	Soft member
Degree of preconsolidation	Greater than weight of overlying sediments and greater than that of semi-consolidated member	Greater than weight of overlying sediments and greater than that of soft member. Preconsolidation is about one-half that of the underlying older bay mud	Normal consolidation
Increase in density on drying	High dry density (stiff to very stiff clay). Density is greater than semi-consolidated member	Marked decrease in dry density (as compared to the older bay mud) at the rate of 10 to 25 lbs. per cu. ft.	Less than 10 lbs. per cu. ft.
Average water saturated weight	131 lbs. per cu. ft.	110 lbs. per cu. ft.	97 lbs. per cu. ft.
Average weight submerged in water	68 lbs. per cu. ft.	47 lbs. per cu. ft.	34 lbs. per cu. ft.
Average moisture content	Less than 40 percent	About 40 percent	Greater than 40 percent
Compressive strength	1,000 to 4,000 lbs. per sq. ft.	1,000 lbs. per sq. ft.	200 lbs. per sq. ft. Strength is a function of pure cohesion and no coefficient of friction exists
Foundation characteristics	GOOD Foundation for piles and other similar structures, and for all but heavy, concentrated load	FAIR Foundation for earthfill having a broad base. Not good for piles or other similar structures	POOR Piles, and other similar structures tend to sink of their own weight
Special properties	----------	----------	Sensitive clay. Swells when wet, tends to desiccate when dried in air[1]
			Highly compressible
			Loses 50 percent of strength when disturbed. Regains strength due to thixotropic action[2]
Origin	Results from flocculation of clay colloids in salt water. Tends to deflocculate when placed in fresh water		
Thickness	Up to 200 feet	Up to 60 ft., 40 ft. average	Up to 70 ft., 60 ft. average
Physical properties and mineralogy	Firm clay with varying amounts of silt and lenses of sandy clay, sand and small-sized gravel. Minus 2-micron sizes are 50 percent normal and hydrated mica; plus montmorillonite, chlorite, diatoms, carbonaceous material, quartz, feldspar. (Schlocker and others 1958)	Plastic, silty clay, and clayey silt, with varying amounts of organic matter, clayey fine-grained sand, and some shell lenses. Clay sizes are 45 to 90 percent of total; ⅓ normal and hydrated mica; ⅓ montmorillonite, and ⅓ mixed-layered montmorillonitic, chloritic, and kaolinitic material. Less than 5 percent carbonaceous material. (Schlocker and others, 1958)	

* Reprinted from Treasher, 1963, p. 15.

[1] The sensitive clay is already in a saturated state, and only when the clay is previously dried and water added will it swell.

[2] Some parts of the bay mud may lose as much as 90 per cent of its strength when disturbed.

during the first 20 years after construction. The settlement varied almost directly with the thickness of the underlying mud (Trask and Rolston, 1951).

The mud is a poor formation for the support of friction piles and it is necessary to carry building loads through the mud to firmer strata below. Engineers attempt to place piles in firm beds below the base of the mud.

The problem of seismic effects upon unconsolidated sediments has been known to geologists and engineers for some time and is discussed briefly in the prior section on earthquakes, and in the accompanying report by Professor Seed. Because of the low strength and high water content, the effects of shaking during an earthquake are more pronounced on the soft muds than on more consolidated sediments or solid rock.

Water-saturated loose sands can become liquefied and flow, even on very gentle slopes, or can subside, compact, and settle.

The seismic effects on bay muds and fills on bay mud are discussed in greater detail in companion reports by Harry B. Seed, and Lee and Praszker. Seismic effects on structures built on bay mud are discussed by Karl Steinbrugge in his report.

GROUND WATER CONSIDERATIONS

Water-bearing formations (aquifers), such as sand or gravel beds, of the adjacent land extend some distance into the bay under a cover of bay mud. These formations constitute a reservoir of fresh water under the salty bay waters.

Ground water investigations are made continuously by the U. S. Geological Survey and the California Department of Water Resources to determine the location and extent of these aquifers. As a result of their studies, it has been shown that land subsidence may occur in areas underlain by dense clay where there has been excessive pumping and drawdown of the water levels. This has already occurred in the vicinity of San Jose. Another effect of excessive drawdown has been the intrusion of salty bay waters into the aquifer, destroying it as a source of fresh water. This has occurred in the Fremont area.

An extensive area of subsidence has been delineated by the U. S. Geological Survey (Poland and Ireland, 1968) in the area southeast of San Francisco Bay. Figure 5 shows that from 1934 to 1967, (1) the ground level in the vicinity of Palo Alto dropped half a foot, (2) the general area including Mountain View, Alviso, San Jose and the southern tip of the bay subsided about six feet, and (3) between Mountain View and

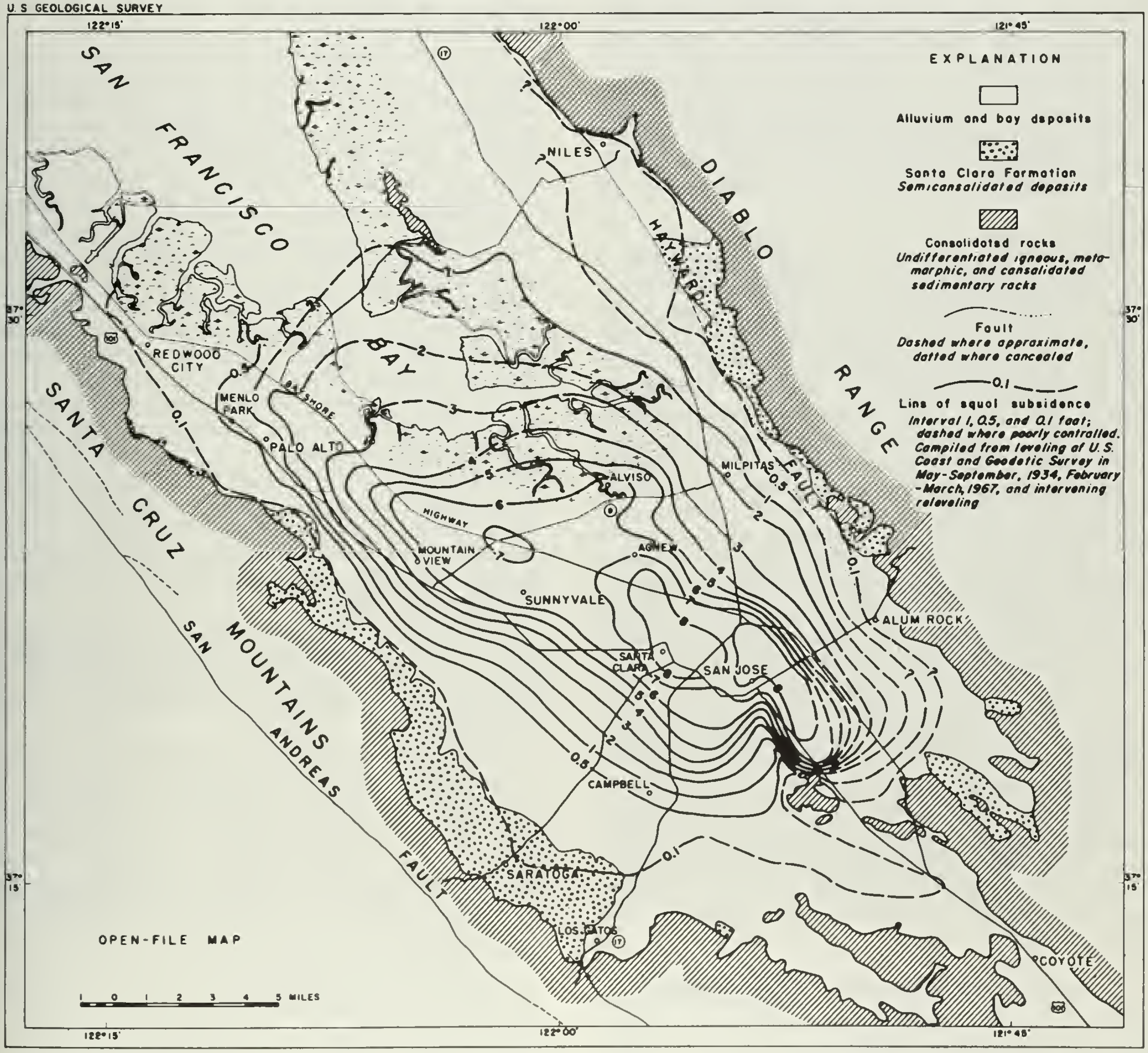


Figure 5. Land subsidence from 1934 to 1967, Santa Clara Valley, California. Source: Poland, J. F., and Ireland, R. L., 1968.

Alviso, and near San Jose a drop of up to seven feet was recorded. Longer term records indicate the area around San Jose has subsided 13 feet since 1912 to the present time. Indications are that if heavy ground water pumping is continued indefinitely in the south bay area, the bay shore in the Alviso area (which has already subsided about seven feet since 1912) could subside as much as seven more feet, requiring extensive dikes to restrain high tides.

Some of the aquifers underlying the bay may be damaged if the mud, which acts as a confining layer over an aquifer, is removed or pierced by such activities as dredging or penetration by bridge piers. Such damage will permit salt water to intrude the fresh water aquifer. If the damaged aquifer contained artesian water, it would flow upward under artesian pressure and perhaps cause subsidence of the surrounding land area.

Data are not available to postulate the inter-relation of ground water depletion, sea water intrusion, and land subsidence. The U. S. Geological Survey has considered a ground water investigation by using geophysical techniques to delineate the ground water aquifers in the bay as an adjunct to its engineering geology studies. Continuing work by the State Department of Water Resources has resulted in the accumulation of basic data. However, data are insufficient to determine the extent and thickness of all aquifers.

While most cities in the bay area are now served by surface water supplies from several aqueducts, it appears that industrial users are tapping ground water supplies more heavily each year. While municipal use of ground water in the south bay has been temporarily lessened by the newest aqueducts, the ground water reservoir there will probably be more extensively tapped in the future as the need for water increases.

No criteria are immediately available to guide decisions concerning dredging or construction in the bay that might penetrate an aquifer. Prospective developers considering dredging or construction that would deeply penetrate the mud layers should be required to consult the State Regional Water Quality Control Board and the State Department of Water Resources concerning possible damage to fresh water aquifers and to provide additional data on ground water conditions in the area to the extent necessary and reasonable in relation to the proposed project.

The other potential problem, salt water intrusion or land subsidence as a result of excessive ground water withdrawal, is even less controllable at the present time. Restriction on water pumping is now possible only through court action by injured parties or through use charges which water districts are authorized to make. More direct controls now appear likely only in the event of severe threat of salt water intrusion or land subsidence when present methods have failed to work.

STATUS OF KNOWLEDGE OF BAY GEOLOGY

Available Published Data

A considerable body of information on the geology of San Francisco Bay exists in the published literature. The prime source (which was used as a basis for this report) is Appendix E of the U. S. Army Corps of Engineers Technical report on San Francisco Bay barriers (1963). Other published reports which are relied upon heavily are "Appendix H, Geology," in the report "Feasibility of construction by the State of barriers in the San Francisco bay system" by the California Division of Water Resources (1955), and articles by Trask and Rolston (1951) and Treasher (1963).

Unpublished Data

The reports mentioned above are quite general and too all encompassing to provide the detail needed for an engineering project at a specific site. There is still a woeful lack of geologic data on the bay.

A major concern of the Bay Conservation and Development Commission must be with geologic problems, such as the depth to bedrock, thickness of mud location of sand, for which data can be obtained only through subsurface methods such as borings or geophysical techniques.

The preponderance of available data are from borings located on the margins of the bay. Most of the borings have been made for engineering projects such as bridges, harbors, vehicular crossings, and salt water barriers. Many of the borings were drilled no deeper than was necessary to penetrate the younger bay mud

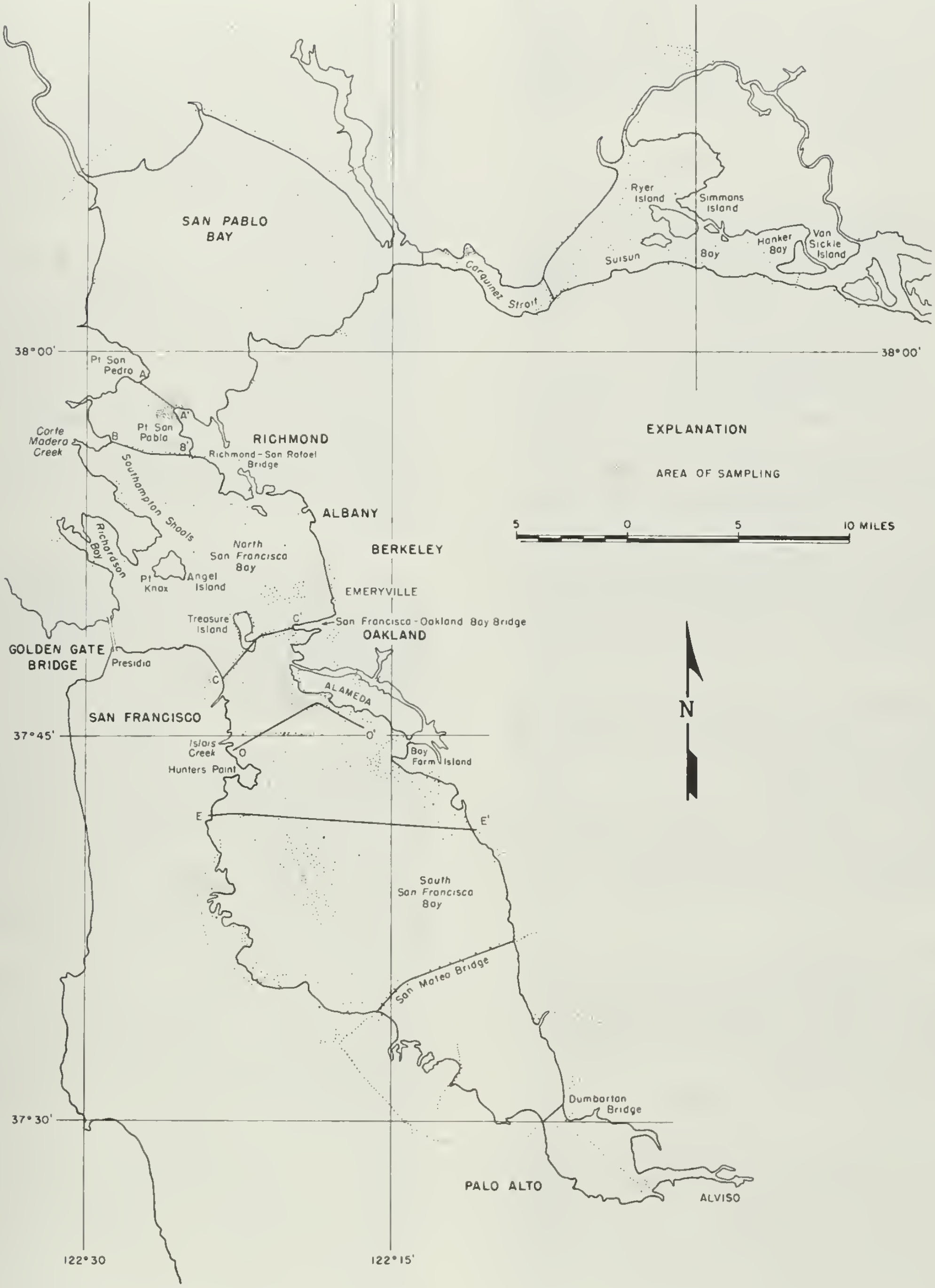
The deeper borings are generally those that have been made in connection with engineering structure built across the waters of the bay and that are founded upon piles driven to bedrock or to firmer sediments Such borings are generally on alignments for the proposed structure and reveal information only along that line. A regional picture is difficult to obtain from these random locations of subsurface information.

The concentration of sampling areas is shown in figure 6. Data from bore holes are virtually absent in San Pablo Bay, except for scattered shallow boring and for water wells around the margins. Thus, firm correlations between sediments in San Pablo and San Francisco Bays cannot be made. Similarly, the subsurface data are too meager in Suisun Bay to permit correlation between the sediments there and those in San Francisco Bay.

The bulk of the drilling records is scattered throughout the files of government agencies and private concerns. In many instances, the obligation to a client for whom the sampling was done prohibits the release of the data.

Information on the type of material on the surface of the bay floor is also quite sketchy. The U. S. Coast and Geodetic Survey attempts to sample from the bay floor as it takes soundings for water depths. However, these are small and therefore inadequate samples Dredge companies prospecting for sand for hydraulic fill have taken surface samples from known sand areas but data as to the depths of sand are lacking, or are kept confidential in company files.

Some geophysical measurements of the configuration of the bay floor and underlying bedrock surface have been made for various purposes such as cable crossings. These profiles are particularly useful when they can be correlated with a nearby drill hole.



ure 6. Sampling areas and location of cross-section profiles. Source: Treasher, 1963.

Research on Bay Geology

Much of the present research on the geology of the y is being conducted by federal agencies: the U. S. eological Survey, the U. S. Bureau of Mines, and U. S. Army Corps of Engineers.

U. S. Bureau of Mines

The U. S. Bureau of Mines is conducting oceanographic research on the feasibility of mining minerals from the sea floor, at its Marine Technology Center at Tiburon. Most of the work is in developing types of equipment to recover minerals from the sea floor. Work has been done in the bay to perfect the equipment. As a corollary to the testing of equipment, the Bureau has attempted to coordinate with the State Division of Bay Toll Crossings, to obtain samples where geologic information is needed. As the equipment is designed for shallow penetration on the surface of the bay floor, this type of research would not substitute for the deeper borings needed for geologic and engineering purposes.

U. S. Geological Survey

The U. S. Geological Survey at Menlo Park has under way on a limited basis a five-year project, entitled "San Francisco Bay Sediments Engineering Geology Studies, California," with an estimated completion date in 1971. The objectives of this study are to evaluate the relations between areal distribution, stratigraphy, origin, geologic history, physical and chemical characteristics of bay sediments, and the response of bay sediments to seismic vibrations and their potential for landsliding and settlement induced by seismic vibrations and other causes.

The study will entail collection and synthesis of available data; acoustical sub-bottom profiling; field mapping; soil tests in the field; sampling from shipboard, obtaining cores 40 feet long in the bay sediments; auger sampling to 100-foot depths inland; and deep coring. Laboratory investigations will be made to determine mineralogy and soil engineering properties.

To date, the program has been in the data compilation stage. The vessel to do the acoustical sub-bottom profiling became available in 1966. Laboratory equipment has been purchased.

If this program proceeds to fruition, the project will fill a gap in our knowledge. Whether the detail needed for specific engineering projects will be provided will depend largely upon the frequency of sampling.

U. S. Army Corps of Engineers

As a continuation of its monumental undertaking for the bay barrier studies, the Corps of Engineers has carried forward hydrologic studies in the bay, but has no planned geologic program.

California Department of Water Resources

The department does not have a project to continue the excellent work which was done for its study of salt water barriers in 1955. However, since the opening of an office in Vallejo, an attempt is being made to compile as much geologic data as are available within its district—which includes the San Francisco Bay region. The original compilation of borings that served as the basis for its geologic Appendix H to the barrier study has been moved to the Vallejo office. As time and money permit, the department will continue to collect and compile data.

SUMMARY

During the geologic events that led to the formation of the present bay, hundreds of feet of soft, relatively unconsolidated sediments accumulated on an old bedrock surface. Engineering problems associated with filling the bay as urban expansion grows, are the result of the poor foundation characteristics of the thick layer of mud or silty clay that underlies the bottom and margin of much of the bay. Variability in the thickness and engineering properties of the mud, as well as the slope and configuration of the bedrock surface, complicates the difficulty in engineering fills

The bay is bounded by seismically active faults and can be expected to experience strong earthquakes. The soft, water-bearing sediments, such as the bay mud may be expected to react strongly to such earthquake-generated shocks.

Sources of hydraulic sand fill are limited to the shoa areas in the bay; the prime source is a large sand bar outside the Golden Gate. Land fill is obtained from a number of quarries that border the bay.

The geology of the recent sediments in the bay is far from completely known. Thickness and extent of the geologic formations (some of which are ground water reservoirs) underlying the waters of the bay are known only through limited boring reports, many of which are highly unreliable, or are held in confidential files.

CONCLUSION

Every day new raw data are developed that pertain to the geology of the bay sediments and their physical and engineering properties. Although the California Department of Water Resources and the U. S. Army Corps of Engineers have accumulated data on a sporadic basis for their barrier studies, there has never been an attempt to collect, compile, evaluate, and analyze these data systematically *on a continuing basis*

Reliability of the data that are available varies almost as much as the natural materials themselves. The difficulty in correlation between bore holes is compounded by the variety of ways in which the materials are described, generally by drillers—not by geologists; and by the absence of accompanying engineering test results.

Generally, the firms that conduct foundation studies in the bay area draw on results from their own file of test borings on prior sites. Reports of the Department of Water Resources and the Corps of Engineer do not provide them with the detail essential for a specific site study.

There is still much to be learned about the fabric mineralogy, and physical chemistry of clays and their relation to engineering behavior. Research is needed in soils engineering through simulated earthquake-loading tests, and changes in strength-on-shearing tests.

The need exists for a state agency to serve as a central clearing house for all the basic geologic data on the bay. The work of securing and compiling, begun by the Department of Water Resources and continued by the Corps of Engineers, should be carried forward A central file of boring logs should be kept, perhaps programmed on computers so that data can be retrieved readily for a specific area. Maps showing the configuration of the bedrock, the thickness of bay mud, and the location of sand deposits, should be kept current as new holes are drilled. There is also a need or an aggressive program to obtain new data by drillng and sampling, and by geophysical methods.

RECOMMENDATIONS

1. A State agency should be designated and funded to ursue actively the collecting, compiling, and analyzing of ata on the geology of San Francisco Bay, on the engieering properties of bay sediments, and on the ground ater conditions. This agency should conduct laboratory esting, core drilling, and geophysical surveys, in cooperaion with other state and federal agencies, particularly the .S. Geological Survey, on a matching fund or participating asis, so that efforts can be coordinated and reinforced.

2. Until such time as such an agency is designated, the ivision of Mines and Geology should be retained to act s the geologic consultants to the Bay Conservation and evelopment Commission on the level of about $20,000 a ear, to conduct studies necessary to provide the engineerg geology data needed to evaluate proposed fill deelopments, and to add new data to that in this report.

3. The Bay Conservation and Development Commission hould require applicants for fill permits to obtain the eologic data necessary for sound foundation engineering esign, including core holes and/or geophysical surveys; nd require that such data be transmitted to the Division f Mines and Geology, where it will serve to build up the ecord and be available for public reference.

4. The Bay Conservation and Development Commission hould request the State Division of Oil and Gas to require n electric log and if available, the core log of the recent nconsolidated sediments in any oil test well drilled in or ear the bay; and such data should be transmitted to the ivision of Mines and Geology.

5. The Bay Conservation and Development Commission hould require applicants for dredging or construction hich deeply penetrates the mud over possible fresh water quifers under the bay to consult the Regional Water uality Control Board and the State Department of ater Resources concerning possible damage to fresh ater aquifers and to provide additional data on round water conditions in the area to the extent necesary and reasonable in relation to the proposed project.

REFERENCES

onillo, M. G., 1965, Geologic map of the San Francisco South quadrangle, California: U.S. Geol. Survey, open file map, scale 1:20,000.

alifornia Division of Water Resources, 1955, Geology, Appendix H of report to the Water Project Authority of the State of California on feasibility of construction by the state of barriers in the San Francisco Bay system.

nkins, O. P., 1951, Geologic guidebook of the San Francisco Bay counties: California Div. Mines Bull. 154.

wson, A. C., 1914, San Francisco, California: U.S. Geol. Survey Geol. Atlas, Folio 193, 24 pp., 15 maps.

uderbock, G. D., 1939, San Francisco Bay sediments: 6th Pacific Sci. Cong., V. II, Proc.; pp. 783–793.

oson, Brian, 1966, Principles of geochemistry, John Wiley and Sons, New York, 3rd edition.

itchell, J. K., 1963, Engineering properties and problems of the San Francisco Bay mud: California Div. of Mines and Geology, Spec. Rept. 82, pp. 25–33.

lond, J. F., and Ireland, R. L., 1968, Land subsidence from 1934 to 1967, Santa Clara Valley, California: U.S. Geol. Survey open-file map.

adbruch, D. H., 1957, Areal and engineering geology of the Oakland West quadrangle, California: U.S. Geol. Survey, I-239.

dbruch, D. H., and Schlocker, J., 1958, Engineering geology of Islais Creek Basin, San Francisco, California: U.S. Geol. Survey, I-264.

Schlocker, J., Bonilla, M. G., and Radbruch, D. H., 1958, Geology of the San Francisco North quadrangle, California: U.S. Geol. Survey, I-272.

Storey, J. A., Wessels, V. E., and Wolfe, J. A., 1966, Radiocarbon dating of recent sediments in San Francisco Bay: California Div. Mines and Geology, Mineral Information Service, March 1966, pp. 47–50.

Trosk, P. D., 1953, Bottom sediments in Inshore survey, San Francisco Bay-Literature survey: Univ. of California Inst. Eng. Research, Technical Report, Series 57, Issue 1, pp. B-1 to B-23.

Trosk, P. D., and Rolston, J. W., 1951, Engineering geology of San Francisco Bay, California: Geol. Soc. Amer. Bull., vol. 62, pp. 1079–1109.

Treasher, R. C., 1963, Geology of the sedimentary deposits in San Francisco Bay, California: California Div. Mines and Geology, Spec. Rept. 82, pp. 11–24.

U.S. Army Corps of Engineers, 1963, Technical report on San Francisco Bay barriers; Appendix E, barrier plans, geology, soils, and construction materials, San Francisco list, 144 pp., 63 fig., 46 pl.

Weaver, C. E., 1949, Geology of the Coast Ranges immediately north of San Francisco Bay region, California: Geol. Soc. Amer. Mem. 35.

SELECTED BIBLIOGRAPHY

San Francisco Bay Geology

Arden, Daniel, Jr., 1961, Sediments from borings along the east side of San Francisco Bay: Unpub. Ph.D. thesis, Univ. of California, Berkeley.

Bonilla, M. G., 1964, Bedrock-surface map of the SF South quadrangle, California: U.S. Geol. Survey open-file map, scale 1:20,000.

California Division of Bay Toll Crossings, 1948, A report to Department of Public Works on additional toll-crossings of San Francisco Bay: 144 pp.

California Division of Bay Toll Crossings, 1951, A report to Department of Public Works on proposed highway toll crossing Richmond-San Rafael Bridge: 233 pp.

California Division of Bay Toll Crossings, 1966, Final report; Preliminary soil exploration for a southern crossing of San Francisco Bay: 17 pp., 47 tables, 5 pl., app.

California Division of Water Resources, 1955, Geology—Appendix H of report to the Water Project Authority of the State of California on feasibility of construction by the state of barriers in the San Francisco Bay system: 62 pp., 9 pl.

Crooke, R. C., and others, 1953, Inshore survey, San Francisco Bay: Literature survey: Univ. of California, Berkeley, Inst. of Eng. Research series 57, no. 1.

Dames and Moore, Consultants, 1965, Report of seismic study, Redwood Shores Project in Report of seismic investigation for Redwood City General Improvement District, No. 1–64: 32 pp., 9 pl., 11 appendices.

Gibson, W. M., Sand waves in San Francisco Bay: U.S. Coast and Geodetic Survey Jour., v. 4, pp. 54–58.

Gilbert, G. K., 1917, Hydraulic mining debris in the Sierra Nevada: U.S. Geol. Survey Prof. Paper 105, 154 pp.

Hoover-Young San Francisco Bay Bridge Commission, 1930, San Francisco Bay Bridge: Rept. Hoover-Young Comm. to the President of the U.S. and the Governor of the State of California, 234 pp., 54 pl., tables.

Institute of Transportation and Traffic Engineering and U.S. Geological Survey, 1950, Selected logs of borings in the City and County of San Francisco, California: Univ. of California Info. Circ. No. 11, 240 pp., map.

Johnson, A. T., Maston, R. P., and Morris, D. A., 1965, Physical and hydrologic properties of water-bearing deposits in subsiding areas of central California: U.S. Geol. Survey open-file report.

Louderbock, G. D., 1939, San Francisco Bay sediments: 6th Pacific Sci. Cong. Proc., v. II, pp. 783–793.

Louderback, G. D., 1951, Geologic history of the San Francisco Bay: California Div. Mines Bull. 154, pp. 75–94.

Mitchell, J. K., 1963, Engineering properties and problems of the San Francisco Bay mud: California Div. Mines and Geology Spec. Rept. 82, pp. 25–32.

Moore, D. B., 1965, Recent coastal sediments, Double Point to Point San Pedro, California; Univ. of California, Berkeley, Hydraulic Eng. Lab., HEL 2–14, 86 pp.

Radbruch, D. H., 1957, Areal and engineering geology of the Oakland West quad., California: U.S. Geol. Survey Misc. Geol. Inv. Map I-239.

Radbruch, D. H., 1959, Former shoreline features along the east side of San Francisco Bay, California: U.S. Geol. Survey Misc. Geol. Inv. Map I-298, scale 1:48,000.

Radbruch, D. H., and Schlocker, J., 1958, Engineering Geology of Islais Creek Basin San Francisco, California: U.S. Geol. Survey Misc. Geol. Inv. Map I-264, scale 1:12,000.

Schatz, B., 1963, A restudy of bottom sediments near the entrance of the Golden Gate: Univ. of California Inst. of Eng. Research Tech. Rept. Series HEL-4-3, 19 pp.

Schlocker, J., Bonilla, M. G., and Radbruch, D. H., 1958, Geology of the San Francisco North quadrangle, California: U.S. Geol. Survey Misc. Geol. Inv. Map I-272, scale 1:24,000.

Schlocker, J., Radbruch, D. H., and Bonilla, M. G., 1954, Preliminary bedrock-surface map of the San Francisco City area, California: U.S. Geol. Survey open-file map, scale 1:24,000.

Schlocker, J., 1962, Bedrock-surface map of the San Francisco North quadrangle, California: U.S. Geol. Survey open-file map, scale 1:18,000.

Sumner, F. B., Louderback, G. D., Schmitt, W. L., and Johnston, E. C., 1914, A report upon the physical conditions in San Francisco Bay, based upon the operations of the U.S. Fisheries Steamer "Albatross" during the year 1912 and 1913: Univ. California Pub. Zool. v. 14, pp. 1–198.

Trask, P. D., 1953, Section B — Bottom sediments ond Section R — Submarine Geology in Crooke, R. C. and others, 1953, Inshore Survey, San Francisco Bay: Literature Survey: Univ. of California Berkeley, Inst. Eng. Research Tech. Rept. series 57, issue 1, pp. B1–B23 and R1–17.

Trask, P. D., and Rolston, J. W., 1951, Engineering geology of San Francisco Bay, California: Geol. Soc. Amer. Bull. v. 62, pp. 1079–1110.

Treasher, R. C., 1963, Geology of the sedimentary deposits in San Francisco Bay, California: California Div. Mines and Geology Spec. Rept. 82, pp. 11–24.

U.S. Army, Corps of Engineers, 1963, Appendix "E"—Barrier plans, geology, soils and construction materials to the Technical report on San Francisco Boy barriers: San Francisco Dist., 144 pp., 63 fig., 46 pl.

Weaver, M. A., and Radbruch, D. H., 1960, Selected logs of borings on the east side of San Francisco Bay, California: U.S. Geol. Survey open-file report, 213 pp.

REGIONAL GEOLOGY

Bailey, E. H., Irwin, W. P., and Jones, D. L., 1964, Franciscan and related rocks, and their significance in the geology of western California: California Div. Mines and Geology Bull. 183, 177 pp.

Bonilla, M. G., 1960, Landslides in the San Francisco South quadrangle, California: U.S. Geol. Survey open-file report, 44 pp., map, scale 1:24,000.

Bonilla, M. G., 1961, City College fault, San Francisco, California: U.S. Geol. Survey Prof. Paper 424-C, pp. C190–C192.

Bonilla, M. G., 1965, Geologic map of the San Francisco South quadrangle, California: U.S. Geol. Survey open-file map, scale 1:20,000.

Books, K. G., 1965, Natural gamma aeroradioactivity map of parts of the San Francisco region, Californio: U.S. Geol. Survey Geophys. Inv. Map GP-483, scale 1:250,000.

Case, J. E., 1963, Geology of parts of the Berkeley and San Leandro Hills, California: Unpub. Ph.D. thesis, Univ. of California, Berkeley.

Clement, W. G., 1965, Complete Bouguer gravity map of the northern part of the San Francisco Boy area and its geologic interpretation: U.S. Geol. Survey Geophys. Inv. Map GP-468, 6 pp. text, map scale 1:125,000.

Hinds, N. E. A., 1952, Evolution of the California landscape: California Div. Mines Bull. 158, 240 pp.

Jenkins, O. P., 1951, Geologic guidebook of the San Francisco Bay counties: California Div. Mines Bull. 154, 392 pp.

Jennings, C. W., and Burnett, J. L., 1961, Geologic Map of California, San Francisco Sheet: California Div. Mines, scale 1:250,000.

Koenig, J. B., 1963, Geologic mop of California, Santa Rosa Sheet: California Div. Mines and Geology, Scale 1:250,000.

Lawson, A. C., 1914, San Francisco, California: U.S. Geol. Survey Geol. Atlas, Folio 193, 24 pp., 15 maps.

Rogers, T. H., 1966, Geologic map of California, San Jose Sheet: California Div. Mines and Geology, scale 1:250,000.

Schlocker, J., and Bonilla, M. G., 1962, Field Trip 4: San Francisco Peninsula: Californio Div. Mines and Geology Bull. 181, pp. 399–406.

Schlocker, J., Bonilla, M. G., and Imlay, R. W., 1954, Ammonite indicates Cretaceous age for part of Franciscan Group in San Francisco Bay area: Am. Assoc. Petroleum Geologists Bull., v. 38. 38, pp. 2372–2381.

Smith, M. B., 1964, Map showing distribution and configuration o basement rocks in California: U.S. Geol. Survey Oil and Gas Inv Map OM-215.

Strand, R. G., and Koenig, J. B., 1965, Geologic map of California Sacramento Sheet: California Div. Mines and Geology, scale 1:250,000.

Taliaferro, N. L., 1951, Geology of the San Francisco Bay counties California Div. Mines Bull. 154, pp. 117–154.

Taylor, S. G., 1956, Gravity investigations of the southern San Francisco Bay area: Unpub. Ph.D. thesis, Stanford Univ.

U.S. Geological Survey, 1966, Aeromagnetic mop of the San Francisc Bay area, California: open file map, scale 1:5,000.

Weaver, C. E., 1949, Geology of the Coast Range immediately nort of San Francisco Bay region, California: Geol. Soc. Amer. Mem. 35 242 pp.

Young, W. R., 1929, Report on salt water barriers below confluence o Sacramento and San Joaquin Rivers, California: California Div Water Resources, Bull. 22, 2 vol., 667 pp., 131 pl.

Earthquakes

American Saciety of Civil Engineers, 1907, the effects of the Sa Francisco earthquake of April 18, 1906, on engineering constructions Am. Soc. Civil Eng. Proc. v. 33, pp. 299–354.

Benioff, Hugo, 1938, The determination of the extent of faulting wit application to the Long Beach earthquake: Seismol. Soc. Amer. Bull. v. 28, pp. 78–84.

Bonilla, M. G., 1959, Geologic observations in the epicentral area o the San Francisco earthquake of March 22, 1957: *in* Oakeshott, G B., ed., The San Francisco Earthquakes of March 1957, Californi Div. Mines Spec. Rept. 57, pp. 25–37.

Bonilla, M. G., 1966, Deformation of railroad tracks by slippage o the Hayward fault in the Niles district of Fremont, Californio Seismol. Soc. Amer. Bull., v. 56, pp. 281–289.

Bonilla, M. G., and Gates, G. O., 1961, Possible earthquake hazard at the site of proposed Foster City, San Mateo, California: unpub report.

Bolt, B. A., and Marion, W. C., 1966, Instrumental measurement o slippage on the Hayward fault: Seismol. Soc. Amer. Bull., v. 56 pp. 305–316.

Byerly, Perry, 1951, History of earthquakes in the San Francisco Bo area: California Div. Mines Bull. 154, pp. 151–160.

California Department of Water Resources, 1964, Crustal strain an fault movement investigation; Faults and earthquake epicenters i California: Bull. 116-2, 96 pp.

Carder, Dean S., 1965, Earthquake investigations in the western U.S 1931–1964: U.S. Coast ond Geodetic Survey Pub. 41-2, 264 pp.

Cluff, L. S., and Steinbrugge, K. V., 1966, Hayward fault slippage I the Irvington-Niles districts of Fremont, California; Seismol. Sa Amer. Bull., v. 56, pp. 257–279.

Coulter, H. W., and Migliaccio, R. R., 1966, Effects of the earthquak of March 27, 1964 at Valdez, Alaska: U.S. Geol. Survey Pro Paper 542-C, 36 pp.

Dalrymple, G. B., and Lanphere, M. A., 1964, Potential earthquak hazards on bay-fill and marshland adjacent to San Francisco Bay Report prepared for the State of California, Bay Conservation Stud Commission, 18 pp.

Duke, C. M., compiler, 1958, Bibliography of effects of soil condition on earthquake damage: San Francisco, California, Earthquake Eng Research Inst., 47 pp.

Gilbert, G. K., Humphry, R. L., Sovell, J. S., and Soule, F., 1907, Th San Francisco earthquake and fire of April 18, 1906, and thei effects on structures and structural materials: U.S. Geol. Survey Bull 324, 170 pp., 52 pl.

Grantz, A., Plafker, G., and Kachadoorian, R., 1964, Alaska's Goo Friday earthquoke, March 27, 1964: U.S. Geol. Survey Circ. 491, 3 pp.

Hansen, W. R., 1965, Effects of the earthquake of March 27, 1964 a Anchorage, Alaska: U.S. Geol. Survey Prof. Paper 542-A, 68 pp.

Iacopi, Robert, 1964, Earthquake country: Menlo Park, Calif., Lan Book Co., 191 pp.

Jaggar, T. A., 1923, The Yokohama-Tokyo earthquake of September 1 1923: Seismol. Soc. Amer. Bull., v. 13, pp. 124–146.

Kachadoorian, R., 1965, Effects of the earthquake of March 27, 196 at Whittier, Alaska: U.S. Geol. Survey Prof. Paper 542-B, 21 pp

Lawson, A. C., ed., 1908, The California earthquake of April 18, 1906 Report of the State Earthquake Investigation Commission: Carnegi Inst. Wash. Pub. 87, v. 1, 451 pp., 146 pl., atlas.

Oakeshott, G. B., ed., 1959, San Francisco earthquakes of March 1957 California Div. Mines Spec. Rept. 57, 127 pp.

akeshott, G. B., ed., 1955, Earthquakes in Kern County, California during 1952: California Div. Mines Bull. 171, 283 pp.

adbruch, D. H., 1965, Approximate location of fault traces and historic surface ruptures within the Hayward fault zone between San Pablo and Warm Springs, California: U.S. Geol. Survey open-file map, scale 1:62,500.

adbruch, D. H., Bonilla, M. G., and others, 1966, Tectonic creep in the Hayward fault zone, California: U.S. Geol. Survey Circ. 525, 13 pp.

odbruch, D. H., and Lennert, B. J., 1966, Damage to culvert under memorial stadium, University of California, Berkeley, caused by slippage in the Hayward fault zone: Seismol. Soc. Amer. Bull., v. 56, pp. 295–304.

ichter, C. F., 1958, Elementary seismology: San Francisco, W. H. Freeman, 768 pp.

ansburn, W. A., 1965, An earthquake reading list—1: California Div. Mines and Geology, Mineral Information Service, v. 18, no. 4, pp. 70–71.

hlocker, J., Pampeyan, E. H., and Bonilla, M. G., 1965, Approximate trace of the main surface rupture in the San Andreas fault zone between Pacifica and the vicinity of Saratoga, California, formed during the earthquake of April 18, 1906: U.S. Geol. Survey open-file map, scale 1:62,500.

ismological Society of America Bulletin, 1963, Special Issue—Oceanographic, geologic, and engineering studies of the Chilean earthquakes of May 1960, 18 articles: Seismol. Soc. Amer. Bull., v. 53, no. 6, pp. 1123–1414.

einbrugge, K. V., Zacher, E. G., Tocher, D., Whitten, C. A., and Claire, C. N., 1960, Creep on the San Andreas fault: Seismol. Soc. Amer. Bull., v. 50, pp. 389–415.

ber, S., 1906, Some local effects of the San Francisco earthquake: Jour. Geology, v. 14, pp. 303–315.

iversity of California, Berkeley, 1964–1966, Bull. of the Seismograph stations: v. 31–33, quarterly numbers.

S. Army, Far East Command, 1949, The Fukui earthquake, Hokiuriku region, Japan, 28 June 1948: U.S. Army, Office of the Engineer, General Headquarters, Far East Command, v. 1 Geology, v. 2, Engineering, 300 pp.

S. Geological Survey, 1964, The Hebgen Lake, Montana, earthquake of August 17, 1959: U.S. Geol. Survey Prof. Paper 435, 242 pp.

hitten, C. A., and Claire, C. N., 1960, Creep on the San Andreas fault (California)—Analysis of geodetic measurements along the San Andreas fault: Seismol. Soc. Amer. Bull., v. 50, pp. 404–415.

llis, Bailey, 1923–1924, Earthquake risk in California, 4 pts.: Seismol. Soc. Amer. Bull., v. 13, pp. 89–99; v. 14, pp. 9–25, 150–164, 256–264.

Miscellaneous

den, J., 1856, Deposit of red sand inside of the bar of San Francisco Bay, California: U.S. Coast and Geodetic Survey Ann. Rept. 1855, app. 20, pp. 162.

Homan, W. J., and Schultz, E. A., 1963, Model tests of shoaling and of dredge spoil disposal in San Francisco Bay: *in* Proc. of the Federal Inter-Agency Sedimentation Conference 1963, U.S. Dept. of Agriculture Misc. Pub. 970, pp. 708–722.

Jennings, C. W., and Hart, E. W., 1956, Exploratory Wells drilled outside of oil and gas fields in California to December 31, 1953: California Div. Mines Spec. Rept. 45, 104 pp.

Jennings, C. W., and Strand, R. G., 1963, Index to graduate theses on California geology to December 31, 1961: California Div. Mines and Geology Spec. Rept. 74, 39 pp.

Lee, C. H., 1953, Building foundations in San Francisco: Amer. Soc. Civil Eng. Proc., v. 79 separate 325, 32 pp.

San Francisco Bay Area Rapid Transit Committee, 1956, Regional rapid transit: 106 pp.

Scott, Mel, 1963, The future of San Francisco Bay: Univ. of California, Berkeley, Inst. of Governmental Studies, 125 pp.

Seed, H. B., and Reese, L. C., 1957, The action of soft clay along friction piles: Amer. Soc. Civil Eng. Trans., v. 122, pp. 731.

Skinner, J. E., 1962, An historical review of the fish and wildlife resources of the San Francisco Bay area: California Dept. of Fish and Game, Water Projects Branch Rept. no. 1, 226 pp.

Smith, B. J., 1965, Sedimentation in the San Francisco Bay system: *in* Proc. of the Federal Inter-Agency Sedimentation Conf. 1963, U.S. Dept. Agriculture Pub. no. 970, pp. 675–706.

Terry, R. D., 1955, Bibliography of marine geology and oceanography, California coast: California Div. Mines Spec. Rept. 44, 131 pp.

Trask, P. D., and Seed, H. B., 1954, Some applications of geology in soil mechanics and foundation engineering: Amer. Soc. Civil Eng., v. 80 (Separate 477).

U.S. Army, Corps of Engineers, 1937, Studies in connection with stability of the hydraulic fill at Yerba Buena Island, San Francisco Bay, California: Tech. Mem. no. 116-1.

U.S. Office of Area Development, 1959, Future development of the San Francisco Bay area, 1960–2020: U.S. Dept. Commerce, 131 pp., 21 maps.

Whitworth, G. W., ed., 1932, Subsidence and the foundation problem in San Francisco: Amer. Soc. Civil Eng., San Francisco Sect., 107 pp.

Williamson, R. S., and Heuer, W. H., 1871, Report upon the removal of Blossom Rock, San Francisco harbor, California, 1870: Govt. Printing Office, 40 pp.

Recent Theses on San Francisco Bay Geology

Conomos, T. J., 1963, Geologic aspects of the recent sediments of south San Francisco Bay: unpub. M.S. Thesis, San Jose State College, 118 pp.

Gram, R., 1966, Marine geology of the recent sediments of central San Francisco Bay: unpub. M.S. Thesis, San Jose State College, 132 pp.

Reese, J. I., 1965, Sedimentation of the Suisun Bay complex: unpub. M.S. Thesis, San Jose State College, 109 pp.

SALT, SAND, AND SHELLS:
Mineral Resources of San Francisco Bay

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SALT, SAND, AND SHELLS:

Mineral Resources of San Francisco Bay

By Harold B. Goldman

INTRODUCTION

San Francisco Bay is an important area of mineral oduction. The mineral commodities produced from e bay include sand, saline minerals (salt, magnesium mpounds, bromine, and gypsum), and shells and y for use in cement. In 1965, the value of these nerals produced exceeded $12 million.

The San Francisco Bay Conservation and Develop- nt Commission has been charged with preparing a mprehensive plan for the orderly development of n Francisco Bay. One of the integral features of plan will be to develop an understanding of the ologic factors that relate to urban development. The ology of San Francisco Bay is discussed in a com- ion report.

The purpose of this report is to provide the Bay nservation and Development Commission with gen- l information on the mineral resources of the bay that it can establish guidelines for future develop- nt in the bay.

The section on shell resources was adapted from a lication by Earl W. Hart (1966), while the section salines was adapted from reports by the late Wil- n E. Ver Planck (1951).

SAND

and is an important source of hydraulic fill in San ncisco Bay, and occurs in or immediately adjacent existing current channels at a number of places be- en the confluence of the Sacramento and San Joa- n Rivers and in the Golden Gate. The largest area f and is on the bay floor between Point San Quentin the city of San Francisco, but deep water pre- les production from much of this area. The general ribution of sand on the bay floor is shown on re 1.

n general, the areas from which sand is being exca- d are operated under lease from the State Lands mission and are the shoal areas: Point Knox Shoal hwest of Angel Island, Presidio and Alcatraz Shoals and southwest of Alcatraz, Southampton Shoal hwest of Point Richmond, and San Bruno Shoal. largest potential source of sand is outside the Golden Gate on the semicircular Golden Gate bar. Sedimentation on the Golden Gate bar has been discussed by Smith (1965).

The long term movement of sediments in the bay has been the subject of a number of excellent works. Gilbert (1917) and Smith (1965) postulated that the Golden Gate bar is composed of sand eroded from the coast. Moore (1965), in a mineralogical study, concluded that the bar was derived from the sand channel in San Francisco Bay west of Carquinez and that longshore transport along this portion of the California coast is only of local importance at the present time.

Other sand deposits that have been dredged, mainly as sources of fill, lie beneath a cover of younger bay mud. The bulk of this sand is on the east side of San Francisco Bay between Point Richmond and Bay Farm Island. This sand lens is dissected in several places by mud-filled channels and may extend southward beyond Bay Farm Island. The amount of information available is too meager to permit an estimate of the quantity of sand available from this deposit.

The depth limit of economic dredging for most projects is currently 45 to 60 feet; the average depth worked on the shoals is 35 feet below the water surface. The most feasible way of mining sand is by suction dredge which loads into a hopper barge. Dredging costs about 30¢ a cubic yard. Cost of hydraulic filling ranges from 60¢ to $1.50 a cubic yard—dredged, transported, and placed. The costs depend upon the amount of materials to be handled and whether or not the disposal area is nearby. Bay mud used as fill ranges from 60¢ to $1.00 per cubic yard placed; sand from $1.00 to $1.50 placed. Estimates of quantities of available hydraulic fill were made by the California Division of Water Resources (1955), later modified by the U. S. Army Corps of Engineers (1963) and are presented in table 1. As these estimates are based upon extremely limited subsurface data, they must be considered as preliminary.

The sands in the bay have a limited potential for higher uses other than as hydraulic fill, such as aggregate in paving mixtures and in concrete, or even for beaches. Approximately five million tons of sand are

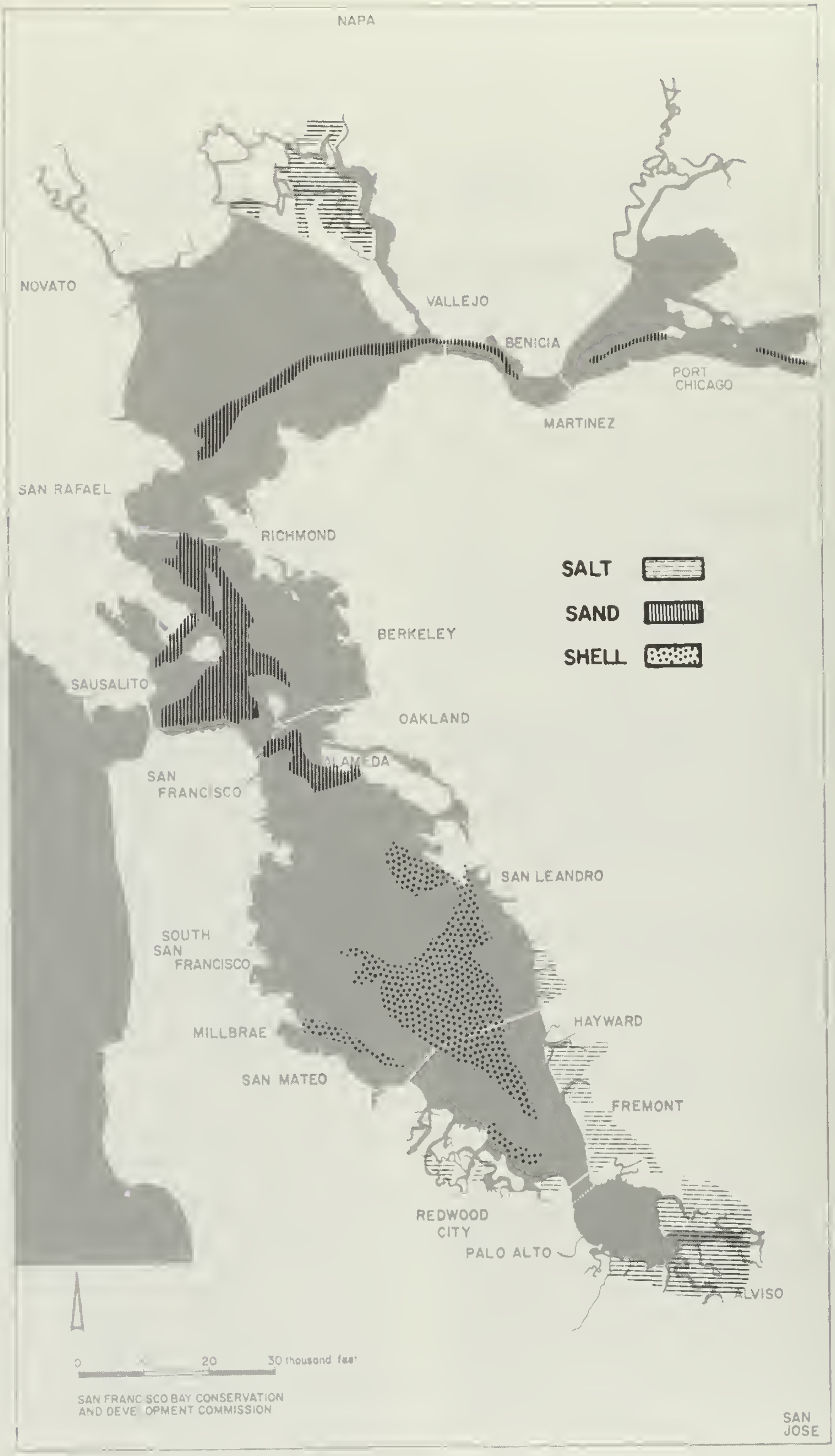

Figure 1. Sand and shell deposits and salt production ponds in San Francisco Bay.

consumed annually in the bay area, primarily in building and paving. In addition, between 350,000 and one million tons of dredged sand is used as fill. An unknown amount of sand is dredged for use on major fill projects, for which no records are available.

The bulk of the sand used in the bay area is tained from pits in stream deposits in Alameda Cou near Fremont and Pleasanton and is trucked or rai to market. Sand for beach replenishment would c from more distant sources such as Felton, Santa C

County, or Seaside, Monterey Canyon. Sand is a low ost commodity that sells for $1 to $2 per ton .O.B. the plant, and as such cannot be transported ny great distances before the cost of transportation quals or exceeds the price paid at the plant. The present truck haul is about 30 miles and may be increased o 43 miles when the closest source is depleted.

The amount of sand in the bay available for future se is not precisely known. The current rate of use oes not suggest that bay sources will be depleted apidly nor that stringent conservation measures are rgent pending a more exact analysis of the supply.

There is no indication of when it will be of ecoomic interest to develop a source of high quality sand the bay. The difficulties in producing such a sand oday are (1) problem of obtaining the proper size radation since the sand at each locality differs in size inge; and (2) the sand must be washed thoroughly nd screened, and the abundant shell fragments reoved. The sands in the bay are predominantly poorly raded (well-sorted), fine-to-medium-grained with a edian diameter of 150 to 500 microns (100 to 40 esh) (U. S. Army, Corps of Engineers, 1963, plate). Pit sand deposits are likely to be largely exusted before bay sources become economically asible.

Table 1. Estimated quantities of sand available for hydraulic fill in San Francisco Bay.*

Location	Cubic Yards (millions)
Antioch to Chipps Island in the Sacramento River near Pittsburg	70
Davis Point to Lone Tree Point, San Pablo Bay near Carquinez Straits	3
Southampton Shoal, San Francisco Bay, offshore of Richmond	2
East bay between San Francisco Bridge and Richmond	60
Point Knox Shoal, San Francisco Bay southwest of Angel Island	10
Presidio Shoal San Francisco Bay, directly north of San Francisco	30
West and south Alameda Naval Air Station, San Francisco Bay	10
West of Bay Farm Island, San Francisco Bay	60
Golden Gate Bar, Pacific Ocean directly outside of the Golden Gate	2,000
San Bruno Shoal, South San Francisco Bay (south portion—1960)	26
Total	2,271

Data adapted from California Division of Water Resources (1955) and U.S. Army Corps of Engineers (1963).

her Sources of Fill

While not a mineral resource in the bay itself, the oining geologic formations in the foothills surunding the bay constitute potential sources of land . There are a number of commercial quarries that ng the bay at which stone is produced for use as fill the marginal lands. The major suppliers of broken ne for use as riprap are at Napa; Brisbane, San Mateo County; McNears Point in Marin County; and Point Richmond in Contra Costa County. Other quarries produce crushed stone; all produce fill material. In 1965, the total production of stone in these counties exceeded 9 million tons.

Another source of fill material arises as a consequence of grading in hills for land development. In San Mateo and Alameda Counties, millions of cubic yards of material have been removed by grading contractors over the years from the foothills, and used on the flat lands below.

Table 2. Estimated quantities of land fill available in the San Francisco Bay area.*

Deposit	County	Maximum cubic yards (millions)
Burdell Mountain	Marin	5,000.0
Sonoma	Sonoma	1,300.0
Big Rock Ridge	Marin	4,700.0
San Bruno Mountain	San Mateo	1,500.0
Coyote Hills	Alameda	80.0
Basalt Rock Co. quarry	Napa	3.0
Basalt Rock Co. quarry	Marin	3.5
Blake Bros. quarry	Contra Costa	1.0
PCA quarry	San Mateo	(with San Bruno Mt.)
Totals		12,587.5

* Data from U.S. Army, Corps of Engineers, 1963.

Nearly all of the geologic formations on the margins of the bay are potential sources of fill when there is an excess of material as a result of cut and fill grading operations; and this would be a major source of the land fill for the bay margins. The amount of material available from commercial quarries would be more expensive, costing from 50¢ to $1.00 a ton, F.O.B. the quarry, and limited in quantity because the future expansion of quarries is rigidly controlled by zoning ordinances. The geological formations that could provide rock fill are shown on figure 1 in the geology report.

It is difficult to determine the amount of material that is economically available as land fill. Estimates made by the Corps of Engineers are presented in table 2.

Most of San Francisco Bay is very shallow. Disregarding problems of placing fill upon mud, it would be at least theoretically possible to fill the bay from the land sources. Rough calculations based on the Corps of Engineers' data indicate 10 billion cubic yards of fill would be sufficient to replace all the water in the bay system.

SHELL DEPOSITS OF SOUTHERN SAN FRANCISCO BAY[1]

Late Quaternary oyster shell deposits, which are exposed over a large portion of southern San Francisco Bay, constitute an important but not widely known mineral resource. Nowhere else in the bays of California are such extensive accumulations of shells known. Although oyster shells from San Francisco Bay were used in a limited way at least as early as 1891 for garden walks and other purposes, there was no concentrated effort to develop the shell deposits until 1924. In that year Agricultural Lime and Compost Company (now Bay Shell Company) began dredging and processing the shells for livestock feed and soil conditioner. The same year, Pacific Portland Cement Company constructed a plant at Redwood City to utilize oyster shells as the basic raw material in cement manufacture. Cement production began in 1925. Thereafter, three other companies commenced dredging shells from the bay and, from 1931 to about 1950, the calcining of shells to make lime became important. The lime was used mainly to produce magnesium compounds by reacting the lime with bitterns from salt works, but a little was sold commercially. Shells are no longer used for lime, but they are still utilized for cement, livestock feed and soil conditioning. As far as the records show there have been only six producers and major processors of shell from San Francisco Bay since 1924. The activities of these operators are summarized in table 3. In 1964, the active shell producers were Ideal Cement Company, Pioneer Shell Company, and South Bay Dredging Company. The entire production of the last company is sold to Bay Shell Company for processing. Since Shellmaker, Inc., ceased operations in 1962 at Newport Bay, Orange County, the above companies are the only shell dredgers active in California.

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[1] Adapted from article by Earl W. Hart (1966).

Occurence of Shells

Even though the shell deposits of south San Fra cisco Bay have been important mineral resources f many years, little systematic effort has been made explore (mainly through drilling and sampling) f new deposits. Consequently, much of the knowled regarding the distribution, character and reserves shells is based on indirect evidence or on informati obtained from studies not directly related to the unde standing of shell deposits. For example, the distributi of shells in the bay floor is known only in a gener way—and this knowledge is based mainly on botto sediment data shown on nautical charts of the U. Coast and Geodetic Survey.

Although the charts do not reveal the subsurfa extent or continuity of the shell beds, they do sho that the shells tend to accumulate with soft mud shallow protected shelf areas of the bay where tid currents are weak. The general location of shell be is shown in figure 1. It seems likely that the shell debi exposed on the bay floor largely represents a rece "population explosion" of the oyster. Presumably, t slightly older, buried shells accumulated under simil circumstances and, to that extent, the surface accum lations provide insight as to the location of subsurfa deposits.

Added information of the subsurface occurren of shells is provided by the hundreds of engineeri borings made in the bay. Unfortunately, the logs borings are of such variable quality that it often

Table 3. Known producers and processors of oyster shells of San Francisco Bay.

Name of operator	Years active	Source of shells	Remarks
Bay Shell Co. (was Agricultural Lime & Compost Co.)	1924—present	Purchased from South Bay Dredging; previously purchased from Beck Dredging; also may have dredged near Alviso and San Mateo Bridge pre-1950(?)	Processes shells for livestock feed an soil conditioning at plant in Alvis (near San Jose).
Ideal Cement Co. (was Pacific Portland Cement Co.)	1925—present	Dredges shells and associated mud east of channel near San Mateo Bridge and barges to plant.	Manufactures cement from shells an mud at Redwood City plant, usin washed shells to "sweeten"; befo 1950 also washed and processe shells for livestock feed and soil co ditioning.
Ortley Shell Co. (W. B. Ortley)	1930–1941	Reportedly did own dredging; location of dredge grounds not determined.	Shells processed for poultry feed Alviso plant.
Pioneer Shell Co. (Capt. L. H. Beck, Beck Dredging Co.)	1931—present	Dredges near San Mateo Bridge east of channel; may have dredged south of Dumbarton Bridge.	Washes shells on dredge and barges t own plant in Petaluma for livestoc feed and soil conditioner; also sells t Ideal Cement; previously sold wash shells to Westvaco Chlorine Produc and Bay Shell Co. and processe shells for commercial sale at pla near Alviso.
South Bay Dredging Co. (Pete Gambetta)	1953 (or earlier)—present	Dredges north of San Mateo Bridge and east of channel; probably dredged south of Dumbarton Bridge earlier.	Washes shells on dredge and sells t Bay Shell Co.
Westvaco Chlorine Products Corp. (was California Chemical Corp.; now is Inorganic Chemicals Div., FMC Corp.)	1931–1948	Purchased from Beck Dredging.	Shells calcined and reacted with bitter to make magnesian compounds a Newark plant; some lime, hydrate lime, and poultry feed sold co mercially.

ifficult to ascertain the amount—or even the presnce—of shells encountered in bore holes. Nevthess, bore hole data show that (1) the shells occur ainly as numerous thin lenses that individually do ot extend very far laterally; (2) shell lenses occur at arious stratigraphic horizons in soft bay mud—but ainly in the upper 30 feet of bay sediments; (3) the reas of surface accumulation shown on the map do ot directly reflect the presence of shells at depth; and 4) buried shell deposits occur in some places outside f known surface shell accumulations.

The principal shell deposits are part of a soft mud nit which contains the youngest sediments deposited n San Francisco Bay.

In addition to the shells associated with soft bay nud, there are relatively pure shell beaches and banks xposed in some tidal areas of San Francisco Bay. hese modern deposits are temporary and small, and herefore are considered to be only minor commercial ources of shells. Sharp decline of the oyster populaon, coupled with wave erosion, is believed to have educed the size and extent of the shell beaches and anks in recent decades. Concentrations of shells in der preconsolidated bay mud also occur, but such eposits appear to be too small and too deeply buried be of commercial value. Broken shells—perhaps deved largely from clams—also are associated with a -foot thick sand zone under bay mud at San Bruno hoal, but the percentage of shells in the sand appears be too small to be of economic interest. However, roken shells associated with sand in the shoal areas of n Francisco Bay north of San Francisco may have ommercial possibilities. It is conceivable that the arse shell debris could be easily separated from the nd by screening.

Composition of Shell Deposits

Virtually all of the shell debris associated with the ft bay mud and the modern beach deposits is deed from the native oyster, *Ostrea lurida* Carpenter, widespread and prolific bivalve. This oyster, which oduces a rather flat and very thin shell, seldom more an 2½ inches long, previously lived in extensive beds the tidal and shallow water areas all around the th arm of San Francisco Bay (see Skinner, 1962, 100, 163, Pl. 3). Increasing pollution in recent cades apparently has so decimated the native oyster pulation that live oysters are now relatively diffilt to find in San Francisco Bay. In addition to sters, the bay mussel and a few other mollusks compse up to a few percent of the shell deposits. The htly worn and broken condition of the shells inates that the shells were transported short distances m their original environment or at least redistribd prior to deposition.

ew analyses are available to indicate the chemical nposition of the shell deposits. However, some idea the chemistry is indicated by a sample composed of washed shells and associated mud obtained from an al Cement Company barge. The sample is a product hydraulic dredging and no doubt is higher in lime (O), 45 percent, than the original in-place sample to the inherent washing effect of the dredging process. Samples which have been washed but are not entirely free of mud, show 54 percent lime. It seems likely that more thorough washing would result in a product relatively high in lime.

Production

It is estimated that roughly 30 million tons of shells have been dredged from San Francisco Bay since 1924. The great bulk of this was used to manufacture cement. Most of the shells were dredged from the vicinity of the San Mateo Bridge east of the main ship channel. Some dredging also is reported to have taken place south of the Dumbarton Bridge. No other shell dredging areas are known to this writer. Recent dredging operations have been carried out principally, if not solely, to the north of the San Mateo Bridge causeway. All current dredgers employ hydraulic methods. Pioneer Shell Company and South Bay Dredging Company work in shallow waters (maximum dredging depth of 10 to 11 feet) and wash shells aboard their dredges. The washed shells are barged to processing plants in Petaluma (Sonoma County) and Alviso (Santa Clara County), where the shells are dried and pulverized for use as poultry and cattle feed or in soil conditioning. Ideal Cement Company operates nearby in slightly deeper water and barges the unwashed shells directly to their Redwood City cement plant. There the shells and adhering mud are used, along with some washed shells and other raw materials, to manufacture cement.

Reserves

Reserves of shells in southern San Francisco Bay are not determinable as information on boundaries, average thickness and content (grade) of the various deposits is not available. However, by making certain assumptions regarding the size and grade of the deposits, some idea of reserves can be obtained. For example, a hypothetical deposit four miles long by three miles wide, with shell beds having an average aggregate thickness of five feet (mud interbeds excluded) and containing an average composition of 70 percent shells, would contain approximately 75,000,000 tons of shells. Although the grade and average thickness of the large shell accumulation exposed on the east side of San Francisco Bay is unknown, its areal extent is much greater than the assumed example and shell reserves may be very large. The deposits along the west side of the bay seem to be considerably smaller in area, but nonetheless may contain substantial shell reserves. If the above assumptions are reasonable, then available shell reserves appear to be quite adequate to support current shell operations for many years to come.

Economics

The shell deposits of San Francisco Bay are actually thin, low grade, unconsolidated to weakly consolidated limestone deposits, that would be far less valuable (or even useless) if located onshore. Some of the advantages enjoyed by the shell operators include: (1) simple inexpensive mining by dredging; (2) ability to beneficiate the shells inexpensively by washing with bay water; (3) use of the clay in the "mud" as a source of raw materials (alumina, silica, iron oxide) in the manufacture of cement; (4) flexibility and low cost in transportation of products to a large marketing area; and (5) low land cost, especially for an urban location.

The disadvantages associated with shell development are not always obvious, but generally relate to multiple use and multiple jurisdiction of the bay. The problems that can develop when dredging, land filling, shipping, sport and commercial fishing, and various recreational activities are all carried on in close proximity to one another are manifold. The varied activities and areas of the bay are managed by a large number of authorities and land owners.

Land ownership in south San Francisco Bay is complex, and mineral rights to large areas are owned both publicly and privately. The State of California owns most of the public land, but some land is held by cities and other government jurisdictions. The second largest private owner, Ideal Cement Company, reportedly controls about 30,000 acres of bay land between Millbrae and Alviso. The approximate distribution of lands owned by Ideal are indicated in Scott (1963, chart opposite p. 4) as lands patented by Surveyor General. Additionally, some state-owned land is leased by Ideal Cement Company for the purpose of shell dredging. Other state leases permit the dredging of sand and gravel.

In most other places outside the bay area, raw materials used in the manufacture of cement are ordinarily obtained from limestone quarries where the rock is drilled, blasted, crushed, and ground. In addition to the bay shells, major deposits of limestone in the bay area are used in the manufacture of cement. These are located at the Kaiser Cement and Gypsum Corporation's operation at Permanente in Santa Clara County, the Lone Star Cement Company's operation near Davenport in Santa Cruz County, and the San Juan Bautista operation in San Benito County. Accelerated depletion of the shell deposits would be reflected in the accelerated depletion of these nearby limestone deposits.

A local supply of raw materials for cement is a critical economic necessity. Cement is a low priced commodity and transportation cost is high in relation to its value, so that cement cannot be transported economically over great distances, except by water. According to Ideal Cement Company, approximately nine million barrels of cement are consumed annually in the bay area. The capacity of their plant using shells is 2.5 million barrels per year. Not only is Ideal therefore a major producer in the bay area, but it is also the closest producer to the market it serves. According to its recent application to the Bay Conservation and Development Commission for a new dredging permit, the company has dredged between 1.5 and 2 million cubic yards of shells and mud per year and expects to increase to 2.5 million yards within the next few years.

Should the oyster shells be lost as a local source of raw materials for use in the manufacture of cement, the possibility of a slight price rise to the consumer exists but hardly seems probable in view of today's competitive market. Nevertheless, in the interest good resource management, this valuable mineral d posit should be conserved and ultimately utilized in planned and orderly fashion. Because most of tl known shell deposits are in shallow waters which a most vulnerable to filling; filling in the areas indicat should be avoided until the shell resource is extracte

The unavailability of the oyster shells would resu in a severe loss of two other operations that rely up the use of crushed shell as poultry grit. The chick growers and the egg producers depend upon tl nearby source of calcium and would have to impc shells from southern California at additional expen

Thus not only the cement industry but also t poultry grit industry would be seriously affect should harvesting of oyster shells from the bay cea

SALINES IN THE BAY AREA[1]

Salines in the bay area are derived entirely from s water. Common salt, magnesium compounds, artific gypsum, and bromine valued at over $11 million, we produced in 1965. The salt refineries of the Leslie S Company are located at Newark, and the ponds f harvesting the crude salt cover the marshes for ma miles along the shores of southern San Francisco B and San Pablo Bay. Inorganic Chemical Division of t Food Machinery and Chemical Corporation also op ates a plant in Newark in which are recovered ma nesium compounds, gypsum, and bromine from b tern, the waste product of salt recovery. These mine products of the sea are sold throughout the northwe

Salt has been produced from sea water since t dawn of history, but San Francisco Bay is one of t few places where age-old methods have been perfect Sea water contains on the average 3.45 percent sal San Francisco Bay is slightly diluted with fresh wat and contains only about 2.8 percent salts. These sa are, in grams per liter: sodium chloride, 21.695; ma nesium chloride, 2.619; magnesium sulfate, 1.842; c cium sulfate, 0.981; potassium chloride, 0.534; calciu bicarbonate, 0.152; and potassium bromide, 0.079.

The great problem is to extract sodium chloride wi a minimum of other salts. Magnesium salts have bitter taste; they are highly deliquescent, as are c cium salts, tending to stay wet.

If a quantity of sea water is evaporated the le soluble salts will, in general, be precipitated first. Wh the volume has been reduced to one-half, calcium ca bonate, magnesium carbonate, and ferric oxide beg to precipitate. At 17 percent of the original volum these materials are completely removed from soluti and gypsum begins to form. When the volume reach nine percent of the original most of the gypsum h precipitated, but it continues to come out down three percent. Salt begins at 8.5 percent and continu to 1.6 percent. Magnesium salts do not come dov until three percent of the original volume, therefo they remain in the bittern, which contains magnesiu potassium, and sodium chloride, potassium bromid and magnesium sulfate.

Throughout the world it has been found mo practical to make the best grades of salt from ro lt and artificial brines, and solar salt manufacture as been abandoned except for local use in the less dustrialized regions. In San Francisco Bay, however, ree factors favorable to rapid solar evaporation are ombined, so that this age-old method has been perected and modernized. First, the dry climate, lack of in during the summer, and summer winds cause high aporation. Second, the thousands of acres of salt arsh provide low-lying land on which the necesrily large acreage of evaporating ponds may be built. he marshes are at or close to sea level which miniizes pumping, and the clay soil provides natural ater-tight bottoms which minimize leakage. Finally, e industries of the bay area constitute a ready arket.

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[1] Adapted from article by W. E. Ver Planck (1951).

The Leslie Salt Company now evaporates bay water ponds covering 50,000 acres. The ponds often look d or brown when seen from a high vantage point. ay water which contains between 1.9 and 2.2 percent lt, is admitted through tidal gates into the first of a ries of 10 concentrating ponds. (Pond areas of 400 500 acres each are required.) Over a period of four five years, the brine is moved from pond to pond. ine in the last concentrating pond contains about .5 percent salts.

After the winter rains, the concentrated brine is run rough a series of crystallizing ponds, where conued evaporation causes salt to form and fall to the ttom. Bittern is drawn from the last pond and sent the bittern storage. An attempt is made to adjust e rate of flow of the brine through the crystallizing nds, so that the brine remains unsaturated with bitn salts. Occasionally, however, in order to prevent e formation of bittern salts, it is necessary to drain e crystallizing ponds and refill them with fresh ncentrated brine. From four to six inches of salt ms during the evaporation season.

Crude salt is harvested from September to late Dember. At the end of the evaporating season, the ystallizing ponds are drained and the crude salt that formed on the bottom is broken up and loaded o cars by machines mounted on caterpillar treads. e crude salt is washed first with concentrated brine then with fresh water before it is stockpiled.

Much salt is sold in bulk directly from the stocke. It contains 99.50 percent NaCl. Semi-refined salt ntaining 99.80 percent NaCl is rewashed, kiln-dried, l screened. For making refined salt, crude material dissolved in fresh water, treated chemically, and porated in a triple-effect evaporator. The crystals med are dewatered, dried, screened, and packed. cuum-refined salt contains 99.9 percent NaCl.

No important use of the discarded bittern was made il World War I, although in 1880 the Union Pacific Company was recovering magnesium carbonate sale to the Hercules Powder Company. It has been smated that the bittern produced in making 100,000 s of salt contains 2,600 tons of potassium chloride, 000 tons of magnesium chloride, 7,500 tons of magum sulfate, and 240 tons of elemental bromine. e shortage of chemicals, especially of potassium s during World War I, created a new interest in ern salts.

When chemicals again became plentiful after the end of the war, these plants operated at a disadvantage. The last one was closed in 1926 and the salt companies have played no further part in the recovery of bittern salts.

The magnesia industry took the next step when in 1923 the California Chemical Corporation, a subsidiary of the National Kellastone Company, built a plant at San Diego for the recovery of magnesium chloride used in the manufacture of oxychloride cement. Bromine was recovered at this plant in 1926, the first time bromine was recovered in California. Later the same year, a second bromine plant was built at San Mateo and this also treated bittern. The California Chemical Corporation then arranged long-term contracts for bittern with the salt companies and began the construction of a much larger plant at Newark to recover bromine and magnesium chloride.

As the market for magnesium chloride was limited, research was initiated on the problem of recovering other magnesium compounds. A pilot plant was built in which lime made from oyster shells was used to precipitate magnesium hydroxide which could be carbonated to make basic magnesium carbonate or calcined to make magnesium oxide. Perhaps the most critical problem was the formation of a granular precipitate which could be easily washed and filtered, but another problem to be solved was the preparation of quicklime from oyster shells. These problems were eventually solved and a new plant was completed late in 1937. In the same year, the California Chemical Corporation passed into the hands of the Westvaco Chlorine Products Corporation.

The first recovery in the United States of magnesium compounds directly from sea water was made by Marine Chemical Company in 1927 at South San Francisco. Relatively small quantities of high-priced compounds such as milk of magnesia, basic magnesium carbonate for coating salt, and magnesium oxide for the pharmaceutical industry, were produced.

Raw sea water, which contains 0.01 of a pound of magnesium per gallon, must be purified before treatment so that impurities in the raw sea water will not appear in the final product. Purification is accomplished by treating the water with chlorine and lime, and filtering.

Magnesium hydroxide is precipitated with a slurry of slaked lime. The precipitate is a gummy slime, which requires special and elaborate handling in order to be dewatered and freed of sodium and displaced with calcium salts. Originally, the magnesium hydroxide was precipitated with calcined shells, but now calcined dolomite from the Westvaco quarry, San Benito County, is used. The magnesium hydroxide may be dried, carbonated, or calcined, depending on the product desired.

The Leslie Salt Company, the largest producer in the bay area, owns all of the 40,000 acres of salt ponds in San Mateo, Santa Clara, and Alameda Counties. In 1961, Leslie started harvesting from 10,000 acres of salt ponds adjacent to San Pablo Bay in Napa County, primarily in response to increased demand for salt and to provide salt to its largest local consumer, the Dow Chemical Company.

In any large scale mining operation, a long range plan of development and exploitation is an essential phase of mineral economics. The production of salt requires long range plans and as it is such an essential industry undoubtedly will be in existence for at least another 100 years, or until such time that mineral economics justify a greater income from a higher land use. One such use, developing the salt ponds for use as real estate, is quite costly and time consuming and requires considerable acreage to be economically feasible.

Former salt ponds owned by the Leslie Salt Company near Redwood City in San Mateo County, are being converted to a high value real estate development, known as Redwood Shores. Approximately 4,500 acres of former salt ponds will be so developed at a great expense for conversion into buildable land for a major urban development housing as many as 60,000 people in the next 25 years.

Future conversion of salt ponds in the bay to real estate will depend in a large part on the pattern of population growth and resultant pressures and demands for usable real estate. According to long range projections of the U.S. Army Corps of Engineers, the greatest population press will be exerted upon the salt ponds in San Mateo County first, then those in Santa Clara County, next Alameda County, and lastly Napa and Solano County. However, spokesmen for Leslie Salt have indicated that the Alameda County and Santa Clara County ponds are not likely to be converted in the foreseeable future—the Alameda County ponds because they are contiguous to the largest single processing plant, which is located at Newark (the most valuable ponds are those nearest the plant, because transportation adds heavily to the cost of salt) and the Santa Clara County ponds, because extensive ground water withdrawal has been causing the general area to subside several feet, making the depth of fill required to overcome the total potential subsidence probably uneconomical.

It appears that salt recovery by solar evaporati will continue as long as the local product is compet tive in price with salt imported into the bay area a there is not, therefore, a more profitable use to made of the land now developed for salt ponds a related facilities. Salt is a low-priced commodity a over-land bulk shipment adds significantly to its cos Within the last few years salt produced in Mexico h been sold competitively in the northwest Unit States and in southern California, but Leslie spoke men report that salt from Mexico is not competiti in the bay area nor is there enough of it to serve t bay area market. So the preservation of salt ponds f salt production purposes is important to the bay ar economy; but total cessation of production is unlike for so many decades that consequences to the loc economy cannot be evaluated and these would appa ently pertain only to the additional cost of transport tion.

REFERENCES

Calif. Division of Water Resources, 1955, Geology, Appendix H of port to the Water Project Authority of the State of California Feasibility of construction by the state of barriers in the San Fra cisco Bay system.

Gilbert, G. K., 1917, Hydraulic mining debris in the Sierra Nevad U.S. Geol. Survey Prof. Paper 105.

Hart, E. W., 1966, Shell deposits of southern San Francisca B California Div. of Mines and Geology, Mineral Information Servi March 1966, pp. 41–45.

Moore, D. B., 1965, Recent coastal sediments, Double Point to Poi San Pedra, California: Univ. of California, Berkeley, Hydrau Eng. Lab., HEL 2-14.

Scott, Mel, 1963, The future of San Francisco Bay. Institute of G ernmental Studies, Univ. of California Press, Berkeley, Calif.

Smith, B. J., 1965, Sedimentation in the San Francisco Bay system: proceedings of the Federal Inter-Agency Sedimentation Conf. 196 pp. 675–708. U.S. Dept. of Agriculture Pub. No. 970.

Skinner, J. E., 1962, An historical review of the fish and wildli resources of the San Francisca Bay area: California Dept. Fish a Game, Water Projects Branch Rept. 1, 226 p.

U.S. Army Corps of Engineers, 1963, Technical report on San Fra cisco Bay barriers; Appendix E, barrier plans, geology, soils, a construction materials, 144 p.

Ver Planck, W. E., 1951, Salines in the bay area, in Geologic guid book of the San Francisco Bay counties: California Div. Min Bulletin 154, pp. 219–222.

BAY MUD DEVELOPMENTS AND RELATED STRUCTURAL FOUNDATIONS

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BAY MUD DEVELOPMENTS AND RELATED STRUCTURAL FOUNDATIONS

By Charles H. Lee and Michael Praszker

SCOPE

Presented herewith are studies, analyses and discus- ons relating to the potential problems of placing fills pon soft bay mud. Specifically, the report presents, om an engineering viewpoint, the problems related fill stability, fill settlement and the response of the nderlying bay mud to its superimposed loads. Settle- ent measurements made on existing fills have been sembled, and future settlement and differential settle- ents for various thicknesses of fill and depth of mud ve been analyzed theoretically. The effect of mud splacement beyond fill boundaries has also been udied from field observations. Recommendations aling with good engineering practice for fill place- ent and fill quality are presented together with basic ncepts justifying such recommendations.

Structural behavior of buildings in mud-filled areas, d the behavior of fills on compressible soils during rthquakes are being presented to the Commission companion reports by Karl V. Steinbrugge, Struc- ral Engineer, and by Professor H. Bolton Seed, re- ectively. It is, however, impossible to separate the bject matter of this report from at least some aspects these two subjects. Wherever pertinent, attention ll be drawn to earthquake problems, but no attempt ll be made to theorize on them.

INTRODUCTION

The behavior of filled-in lowlands underlain by bay ud has been a matter of concern to both laymen d experts in the San Francisco bay area ever since velopment of such lands commenced. Bay fills date ck to the latter part of the 19th Century, the first a to have been filled being the San Francisco water- nt. The extent of bay filling that has taken place ring the last three decades has far exceeded that of preceding 80 years.

The history of early bay fills has been recorded by ious agencies. Some of its marks in the form of pidated buildings, continuously sinking streets and tured utilities are still extant. Evidences of destruc- n wrought upon certain bay-filled areas during the earthquake of 1906 have long been covered up, and subsequent refillings to bring streets up to grade are also a matter of the past. Some kind of stability seems to have been reached during the 100 years' existence of these fills, but in some areas the present degree of this stability is a matter of speculation.

It should be pointed out that the very early fills were placed haphazardly, and it is doubtful whether any attempt was made to predict their future behavior with respect to either total and differential settlement or even static stability, let alone behavior during earthquake.

Although recent fills have been placed with the aid of available technological skill, there still exist some uncertainties with regard to the ultimate behavior of these fills under static conditions or during earthquakes.

All fills placed on mud are subject to subsidence due to the compressible nature of the mud, but the amount of subsidence and especially the pattern in which it develops can be predicted only under assumed ideal conditions. Under such conditions it is assumed that the mud is progressively compressed downward until stability is attained. The time during which subsidence continues and the amount of subsidence is then computed on a theoretical basis. Such consideration does not account for subsidence due to lateral displacement of mud, nor could the amount of such lateral displacement be predicted on a rational basis. During the process of filling, particularly on the uncrusted bay mud, the intrinsic characteristics of the mud are likely to have changed, and by the time the fill is fully placed the underlying mud has lost most of its initial strength and has become an uncontrollable jelly-like mass which may flow in any direction. Upward displacement of the bay bottom far beyond fill boundary has been observed, and while there is no theoretical means by which the degree of such displacement can be computed, it invariably appears in the form of mud waves which sometimes extend more than 1,000 feet into the open bay.

There appears to be some reassurance in the fact that fills placed over encrusted tidal marshes have proved to be more successful with regard to lateral mud movement. There have also been cases of judicious methods of fill placement whereby resulting mud wave action has been minimized. However, future behavior of even these fills, both with regard to effects beyond fill boundaries and during earthquake, remain to be ascertained and cannot be predicted with certainty at this time.

The casual observer looking north during low tide at the east bay toll plaza of the San Francisco-Oakland Bay Bridge, can actually see the mud heave which took place far beyond the fill boundary. Similar observations can be made from the Candlestick causeway and at Candlestick Park parking lot. Some of these heaves were intentionally created; nonetheless, the lateral movements of mud underlying extensive fill may encroach upon navigation channels as well as the open bay.

To a large extent, it is the lateral mud flow from under fills which accounts for observed settlements that by far exceed the theoretically predicted settlements, as is the case in some fills described in this report.

Problems have developed to various degrees relating to structures supported either directly by fill overlying mud, or by deep foundations. The problems range from tilted buildings, cracking of walls, and vertical separation of buildings, to the sinking of surrounding ground in the case of piled foundation.

Where buildings are constructed on fills newly placed on mud, due consideration must be given to types of foundations in relation to the future utility of the buildings. If foundations take support in the fill overlying the mud, then the foundations will settle as the fill settles, even if the effect of foundation loads is neglected. Since uniform fill settlement cannot be anticipated, differential settlements in the building are inevitable. For buildings occupying an extensive area on fill 10 to 15 feet in thickness overlying 50 to 60 feet of mud, such differential settlements may be in the range of six inches to a foot. Some warehouses in the bay area constructed over bay mud have undergone settlements in this range.

When dealing with high-rise buildings supported on deep piles, or any other heavy structures supported on piles, the problem, apart from the stability of such foundations during earthquakes, still remains with regard to the surrounding ground which carries utility pipes such as sewer, water and gas. One case history will show surrounding ground settling away from building to an extent that ramps are used for entrance to the buildings.

This report presents recommendations for practical support of buildings on fill with a view to minimizing differential settlements by allowing the fill to undergo its most critical subsidence prior to construction of the buildings. It also deals with the practical aspects of deep foundations with emphasis on future behavior and avoidance of deterioration.

The purpose of this report will be served if it brings about a clear understanding of what is entailed by filling soft land, and to what degree such operations may be successful, at best, and at worst, what failures may result by injudicious operations.

THE NATURE AND CHARACTERISTICS OF SOFT BAY MUD

The soft bay mud beneath low lying lands on t
periphery of San Francisco Bay is generally of mari
origin. The history of its deposition is immateri
within the scope of this report, but its composition
important for the purpose of predicting its behavi
under superimposed loads.

The soft bay mud consist of fine-grained particl
ranging from clay size (less than 2 microns in dia
eter) to silt and very fine sand (less than 100 micro
in diameter). These soils have been deposited by sed
mentation in still waters and have accumulated duri
the past as the sea level progressively rose toward t
end of the last glacial period. Included in the depos
tion of this silty clay are various organic substanc
such as shells, vegetable matter and peat. Also includ
are lenses and stringers of fresh water deposits such
sand and gravel, and occasionally there are sand ba
overlying the mud.

Since the progressive buildup of the organic clay
silt, generally referred to as “bay mud,” took pla
over a long period of time, it has buried the previous
existing ground surface with its open valley and ravin
extending back into the hills. The rise of the oce
level caused inundation of the valley and high
ground with resulting deposition of bay mud and t
filling up of ravines. The configuration of the b
bottom and its existing shores is therefore highly i
regular and the thickness of superimposed mud vari
greatly.

The bay mud that is exposed at low tide and al
that which is submerged in the bay, is in an unconso
dated, semi-fluid state; that is to say, it has not unde
gone consolidation to the extent that have the old
bay muds underlying the bay at greater depths. Pare
thetically, the older and lower bay muds are separat
from the upper, unconsolidated bay muds by a thi
layer of alluvial deposits and will not be consider
in this report.

The upper layer of bay mud lying above mean ti
and especially above high tide, is largely derived fro
fine sediment deposited from local drainage wate
Its pore water is brackish. It supports marsh veget
tion which forms a mat of roots and fiber often a
cumulating as peat. During the summer periods it dri
out by evaporation and tends to consolidate; a me
brane or crust of 2 to 3 feet thickness has thus be
formed and covers the soft mud. Tidal lands original
exposed at low water when levied off and used f
agriculture tend to form a similar crust after years
use. Such a crust acquires tenacity and, if unbroke
will support light grading equipment. It can be qui
useful in grading operations if preserved in its u
broken condition.

Dry unit weights of the upper bay mud typical
vary from as low as 40 pounds per cubic foot to
pounds per cubic foot. The voids in the bay mu
which are the spaces intervening between solid pa
ticles, are roughly twice the volume of the sol
particles. Thus, a cubic foot of bay mud is made
of one-third of a cubic foot of solid particles and tw

irds of a cubic foot of intervening space which is ccupied by water or air.

Due to the fine texture of its grains and the jelly- ke matrix, bay mud is known to be highly imper- eable. Impermeability can be illustrated by the diffi- ulty with which water can be squeezed out from the oids, with accompanying decrease in volume. It is ommon knowledge that clays, once saturated, can nly be dried over a long period of time by squeezing ut the water; contrasted to which is the behavior of ranular sand which will readily give up its water. t is this phenomenon of relatively high impermeability f the bay mud that gives to it its low strength and its ncapacity to support appreciable loads.

In its pristine condition bay mud has reached a state f "continuing equilibrium," by which is meant that he natural process of consolidation under its own eight is approaching an asymptotic limit. The term onsolidation simply implies a decrease in volume of a oil body by virtue of reduction of its voids which re filled with water and/or air. In order to decrease he volume of the voids, it is necessary to force the ater and/or air out of them. Doing this is equivalent filling a rubber balloon with water, puncturing tiny oles in the balloon and letting the water escape. If he holes are very small, it will take a long time for he water to escape by its own action. An applied xternal pressure will help the process and accelerate . The voids in bay mud can be visualized as a large umber of water-filled tiny balloons, with infinitesimal oles through which the water can escape. Even when lens of fibrous peat or clean sand is embedded in a ody of mud, the water which occupies the lens can- ot freely escape but must percolate through the sur- unding mud. Such a water-filled lens may be viewed a single large balloon, similar to the ones described ove.

THE MECHANICS AND BEHAVIOR OF BAY MUD UNDER SUPERIMPOSED LOADS

When a load such as fill is placed upon mud, the ater in the tiny voids is subjected to a corresponding crease in pressure and the water is squeezed out at a gher rate than under natural conditions, just like ueezing a water-filled balloon punctured with tiny les. If the pressure is not applied too rapidly and not too great, a certain amount of water will squeeze t in a certain interval of time and the volume of the lloon will correspondingly decrease. If the pressure great and is applied rapidly, the balloon will rupture cause the water cannot escape fast enough and the tire pressure is directly transmitted to the water and nce to the balloon. In this case the tiny holes in balloon might just as well not be there, since prac- ally no water will escape in the short interval of essure application.

It can be easily visualized that upon the application a superimposed load on mud, the entire load is mentarily transferred to and carried by the en- pped water. If the load is not too great and the plication is slow, the water will manage to escape ough the tiny pores and the solid grains will be ced to come closer together until they are in a position to carry some or all of the superimposed load. But if the load is great and it is applied suddenly, the high pore water pressures set up in the voids inhibit the soil from developing the shear strength required to prevent failure.

The above analogy is offered to explain in simple terms what has been known to the profession for a long time, namely *The Load-Carrying Capacity of Bay Mud Depends Upon the Rate of Loading*. The appended case histories of experience with bay fills clearly point to the cause of failures due to too rapid loading. In common terms, the mud upon loading *Bulges or Produces Mud Waves* while the filled areas *Sink*. The mud is said to be in a state of plastic flow.

EXPECTED TOTAL AND DIFFERENTIAL SETTLEMENTS OF FILL PLACED ON MUD

The rational approach to determining the future settlement of fills on bay mud does not include the consideration of:

(a) Settlement within the fill body itself.
(b) Plastic flow of the supporting mud under load.
(c) Lateral movement of the mud resting upon an inclined plane of rock or firm soil.

It is assumed that settlements, as computed by available technological means, merely represent an orderly theoretical expulsion of pore water from the mud body under the action of the imposed fill loads. If these conditions are not met in the field, i.e., if any of the above conditions (a), (b), or (c) occur, then the theoretical computations are either appreciably vitiated or totally voided.

The amount of settlement to be expected under ideal conditions in fills newly placed on bay mud is a function of:

1. The compressibility of the mud, C_c.
2. Depth of mud, d.
3. Density of fill, γ
4. Height of fill, Δh

This function may then be expressed in a simplified manner:

$$S = K\ (\Delta h)\ (\gamma)\ (d)$$

where "S" is total settlement, and "K" is a dimensionless constant depending upon various properties of the mud itself, including "C."

It is not necessary to understand the mathematical relation stated above, other than to realize that the total settlement, under ideal conditions, will depend upon all these parameters. If then a blanket of fill of uniform thickness is placed over mud, the settlement will be uniform if the thickness of the mud is uniform; all other factors being equal. If the mud thickness is not uniform, then differential settlement will result from this cause alone. Also, if the fill thickness varies, then additional differential settlement will result due to this cause.

It can now be appreciated that only under ideal conditions can uniform settlement be obtained when large mud flat areas are developed by placement of fills, since the mud composition as well as the mud bottom varies considerably due to many geological factors. Frequently the mud rests upon a sloping surface, particularly when located close to the existing bay shore. This can be explained in terms of the geological formations underlying the bay. Consequently, when fill is placed on offshore mud flats the following conditions must be taken into consideration:

1. Variation in mud thickness within the fill area.
2. Degree of slope of the mud bottom.
3. The amount of plastic flow that may develop during filling.

If any of these three conditions are found to obtain in the field, deleterious results may be prevented only by special engineering features to be included in the design. Suggested provisions to be included in site development to overcome such conditions involve methods and rates of filling, preceded by thorough soil explorations.

As a guide for expected settlements under ideal conditions, figures 1 and 2 are presented. Figure 1 shows the relation between expected settlement for various fill and mud thicknesses. Figure 2 shows percent of settlement (consolidation) versus time in years, with mud thicknesses as parameters. Figure 1 assumes moist unit weight of 110 pounds per cubic foot. Water table elevation of 0.0 mean sea level and top of fill elevation of 11 feet above Mean Sea Level are also assumed. Both assumptions are reasonable for the case of sand fills over submerged bay mud. The charts are representative of the well established characteristics of th upper bay mud in this region.

To illustrate the use of the charts, suppose it is required to read the ultimate settlement produced b 10 feet of fill underlain by 40 feet of bay mud. Figur 1 will show the ultimate settlement as being 4 fee Figure 2 indicates that 48 percent of this ultimat settlement will take place in the first five years, i.e 1.92 feet. Sixty percent of this settlement will be completed at the end of 10 years, that is, 2.4 feet; th remaining 40 percent, or 1.6 feet of settlement, wi take place in 90 years thereafter.

These theoretical considerations, together with settlement measurements in the field over a number c years, lead to the conclusion that for fills over muc less than 40 feet in thickness, the most severe settlemer takes place within the first five years after fill placement. Although actual settlement continues beyon this time, the rate at which the mud consolidates much reduced.

For mud thicknesses in excess of 40 feet, periods i excess of five years will be required for the most critical settlement to take place. Figure 2 shows that fc mud thickness of 60 feet, only 35 percent of the ultimate settlement will take place in the first 10 year Consequently, when major fills are placed over dee mud, much research is needed to determine fill settlement behavior as a function of time and the poss

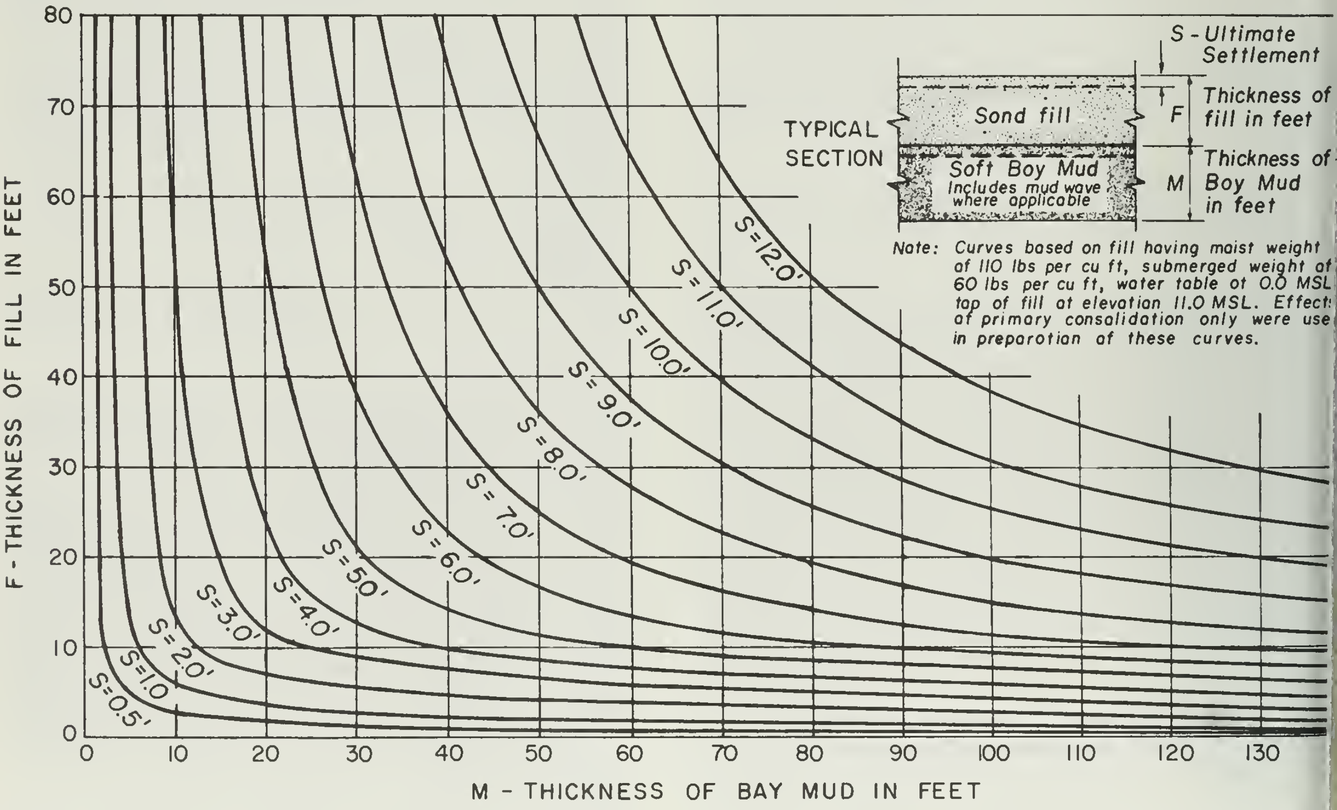
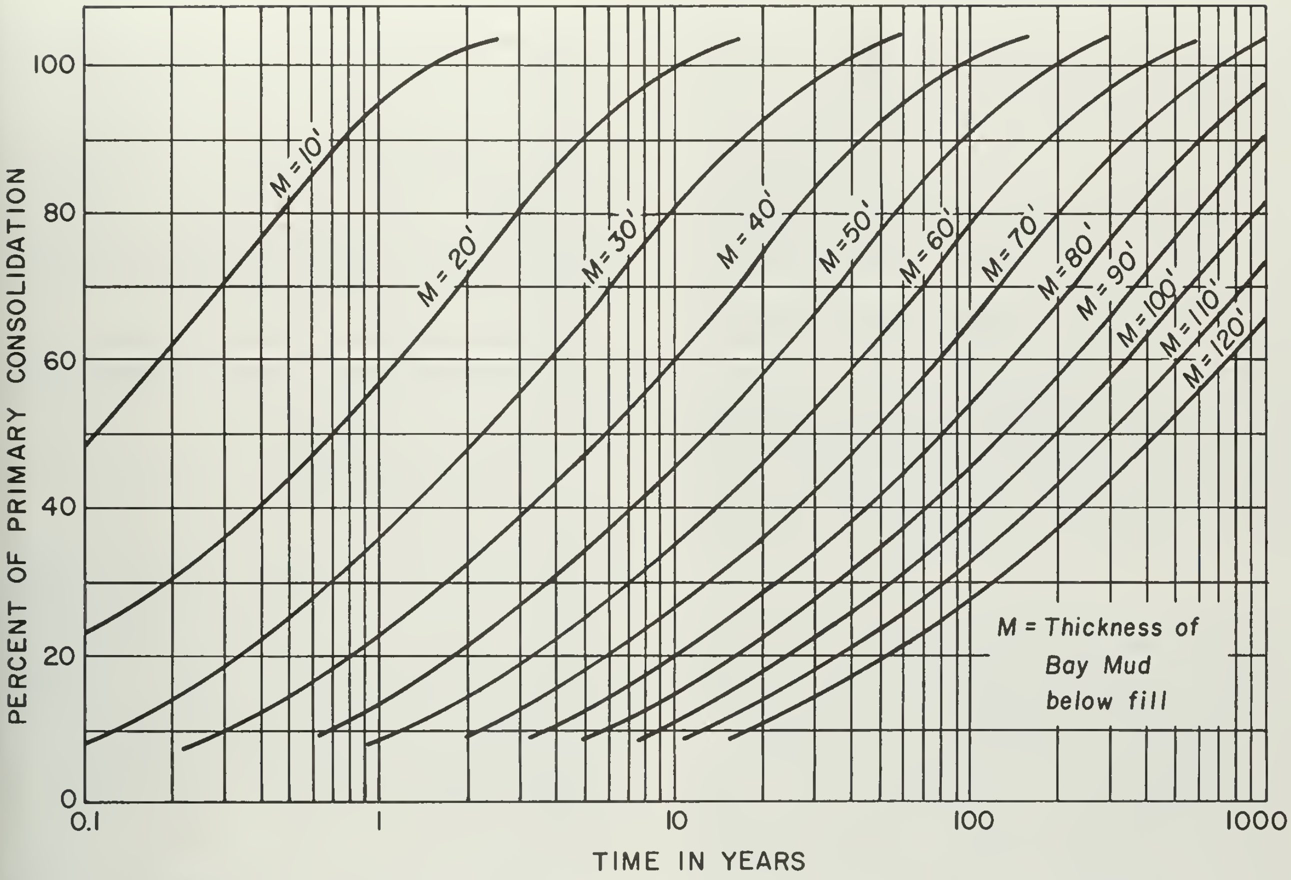


Figure 1. Ultimate amount of settlement of fills according to thickness of fill and thickness of underlying bay mud. Source: Adapted from U. Army, Corps of Engineers, Comprehensive survey of San Francisco Bay and tributaries, Appendix E, figure 23.



gure 2. Percent settlement of fills over time according to thickness of mud. Source: adopted from U.S. Army, Corps of Engineers, Comprehensive survey of San Francisco Bay and tributaries, Appendix E, figure 24.

ility of the use of various devices to accelerate settle-
ients before construction is commenced.

As a practical means of establishing the pattern of
ettlement in any area of fill on bay mud, it is recom-
iended that a number of settlement markers be estab-
shed in such a manner as to embed the markers in
ie mud before filling commences, the top of the
iarkers to be placed above the final fill elevation.
levations of the top of these markers must be estab-
shed prior to filling, during filling, and at regular
itervals after the fill is placed. When settlement
ersus time has been observed, a curve can then be
rawn indicating the settlement trend with respect to
me. The observed settlement will also give a reliable
idication or verification of the variation in mud thick-
ess. If total settlements at various points after a num-
er of years are then plotted in plan, a settlement
ontour map will be established. The configuration and
iape of the settlement contours under ideal conditions
ill be a replica of the contours of the mud bottom.

The practical application of the information thus
btained will enable the developer to adjust for differ-
itial settlements wherever they occur, and may also
ad to further investigation of very irregular settle-
ent behavior.

A process called "surcharging" is sometimes used to
duce the amount of settlement which the completed
l will experience. A larger amount of fill is placed
an is ultimately intended on the site. After enough time has passed for the water to be squeezed out of the mud, the excess fill is removed. The removal of the surcharge will leave the mud in a preconsolidated condition and further consolidation thereafter will be largely reduced if not entirely eliminated, depending upon the duration of the surcharge and other factors.

METHODS AND CONTROL FOR PLACING FILL ON BAY MUD

From the previous description of the composition of the top layers of bay mud, it should be evident that we are dealing with a semi-viscous material similar to jelly which can easily change its geometrical configuration. In fact, the loading capacity of bay mud is not measured so much by the load it can support as by the deformation that such a load may cause. When a load upon bay mud is confined to a small area, the load will make an imprint in the form of a permanent set, provided the load is not large enough to cause the mud to push out laterally and, consequently, bulge at the periphery of the loaded area. The problem, then, when loading bay mud is mostly the prevention of lateral flow of the mud under the load. If hypothetically it were possible to confine the loaded area laterally to an appreciable depth, then the only effect of loading mud would be vertical deflection. The load-carrying capacity of the mud under these conditions would be practically infinite, but in practice such lateral confinement, although possible from an engineering standpoint, may be unfeasible economically. Consequently, when footing loads are applied directly to the mud, the allowable bearing capacity must be kept low, say 300 pounds per square foot, thus holding deformation to a minimum. Some lateral deformation however is practically inevitable. Depositing fill on bay mud constitutes the application of an external force or load upon the mud. If a certain blanket of fill could be placed on top of the mud in such a manner as to spread the load uniformly over the whole area at one time, then this would be equivalent to placing a large footing upon the mud. But this procedure is not practical. In actual fact, fill material is placed by end-dumping upon a certain limited area and then spreading stage by stage, so that while a certain area has been loaded, the neighboring area at the edge of the fill is free to react plastically due to the adjacent load. The observed effect of such a procedure is the creation of mud waves in front of the loaded area.

From a mud wave point of view, the ideal method of placing fill on bay mud is by means of hydraulic dredge whereby the dredger fluid flows over large areas and the building up of the sediment progresses more or less uniformly and slowly. This, if done judiciously, prevents the occurrence of mud waves.

The continuous maintenance of flat slopes at the edge of fill will greatly reduce the size and creation of mudwaves, provided, however, the fill is not placed too rapidly and its final height is limited. Slopes of 10:1 (10 horizontal to 1 vertical) are known to be satisfactory, and it is important that such slopes be maintained during the process of filling, which is difficult to achieve unless the fill is placed hydraulically and the rate of filling does not exceed three feet per year.

There have been many instances in the bay area and elsewhere where attempts have been made to deposit layers of large stone upon cut mud slopes to provide erosion control. What may have been in the minds of the engineers who attempted such injudicious filling of the mud is not known, but in view of the foregoing description of the makeup and behavior of bay mud, it is beyond the realm of wishful thinking that such fills could be permanently stable. Records show that these rock fills disappeared suddenly into the mud and only a mud wave was left to indicate the burial ground of the rock.

In other cases, fills have been placed to appreciable heights with no regard to the significance of mud waves that were produced during the filling operations. As a consequence, a longitudinal section through the filled area after fills had been placed showed the contact plane between the mud and the fill to be a continuous warped surface. While the designed thickness of fill was to have been uniform, say 20 feet, the actual conditions of the fill show variation of fill thickness from a minimum of, say 10 feet, to a maximum of, say 30 feet. Under these conditions of filling and final placement, there is no method of computing, or even of guessing, as to what the final settlement pattern might be.

It is therefore concluded that for a satisfacto
placement of fills on mud for which settlement co
putations have been made under ideal conditions, t
following requirements should be met in the field:

(a) At no time should a mud wave be allowed
develop during filling operations. (This may neces
tate in some cases a filling rate as low as three fe
per year.)

(b) Boundary conditions should be strictly co
trolled to prevent progressive outward lateral flow
the mud.

Practical means of accomplishing these requireme
might be the placing of the first blanket as hydrau
fill of a minimum thickness of three feet before co
paction equipment is put in use.

CRITICAL BOUNDARIES OF INSTABILITY UNDER STATIC CONDITIONS

The balancing of a fill blanket on soft bay mud
by no means to be viewed as being statically stab
Even if the blanket has been uniformly placed, a
mud waves have been prevented during placing, t
stability of the boundaries still has many indetermina
aspects. A large fill blanket normally constitutes
area of infinite dimensions, so that each element
area in the center of the blanket is protected by co
fining pressures from its neighboring elements. Wh
approaching the boundary of the fill blanket, cond
tions change rapidly. At the very edge of the f
blanket the mud is free to move outward and upwar
The upward movement due to plastic flow may
prevented by a gently sloping embankment, as me
tioned earlier, but only to the extent of the confini
downward pressures that the embankment affords. T
actual upward pressures at the periphery of a f
blanket do not lend themselves, at the present sta
of technology, to mathematical analysis. From obse
vations it has been established that some heaving b
yond the fill periphery is inevitable; it is more pr
nounced in the case of fills with steep slopes than wi
gentle slopes. It is also more pronounced when fi
have been placed by end-dumping rather than
hydraulic dredging.

One further element of boundary condition th
usually causes instability is tidal fluctuation. At a ce
tain stage of high tide the slope of the bay fill
partially confined by the tidal water. At other stag
during low tide this confinement is removed. The f
slope and its underlying mud is therefore subjecte
to a continuous process of stress change, and the u
derlying mud may fail without any other extern
provocation.

When fill has been placed on mud which rests upo
an inclined plane, there is a constant tendency for t
loaded fill to creep downward. After the lapse of som
time, two typical boundary conditions may prese
themselves; the boundary closest to the shore ma
show cracking of the fill whereas the offshore boun
ary will show heaving of the mud beyond the fi
The mud therefore is undergoing slow lateral di
placement along the inclined plane and carries t
fill with it.

The term “boundary condition,” as used in this ontext, also applies to any abrupt irregularities in the ud surface such as would be created by injudicious ethods of filling. If, for instance, the fill has been laced in such a way as to remold the mud surface form crests, which are close to the fill surface, d adjacent valleys, which have depressed the mud lastically, then each crest and valley constitutes a cal boundary condition. The plastic deformation of e mud in this fashion reduces the normal strength haracteristics of the mud and constitutes a potential eak zone. Although the fill overlying this irregular ud surface may balance itself under normal con- itions, it is likely to be the first area to rupture under dverse conditions.

In some instances, particularly when deep water is equired just outboard of the fill, dikes or sheet pile t-off walls are used as a fill boundary. The Army reet Terminal in San Francisco and the new 7th reet Terminal in Oakland are examples of this kind construction.

All the above mentioned conditions become more itical in the case of earthquakes, which are not con- dered in this report.

The stabilization of boundary conditions is there- re deemed essential to achieve nominal stability. The e of deep sand trenches has been mentioned but her stabilization methods such as effective surcharge, ainage by filters, etc., are presently undergoing test- g. Much research is needed to establish a practical d economic method to provide boundary stabilities bay fills.

It should be noted that the above discussion of undary instability of fills placed upon mud applies imarily to deep mud deposits or to mud resting upon clined firm downward sloping surfaces. Where the ud is shallow or rests upon firm upward sloping rfaces at moderate depth, stability conditions are odified and plastic flow with heaving may not be a rious problem. The critical depth of mud which ntrols boundary conditions with respect to lateral astic flow is not known and should be the subject research.

TYPES OF FILL THAT CAN BE USED FOR FILLING SOFT LAND

The term “fill” or “fill material” generally relates t solid, particulate material that can be deposited upon m or semi-liquid ground with resulting rise of the efilled ground surface elevation. Requirements such a nonperishable, noncompressible, granular or well- aded materials have to do with fill quality that is quired for any specific purpose. It therefore follows at almost any material which is capable of raising e ground surface of an area upon which it is de- sited is fill material.

Fill materials that may be used for filling soft land ch as bay mud include almost all fill types, except, rhaps, coarse rock or boulders which have the tend- cy to sink into the mud without appreciably raising e surface area, or, more often, the area outside the ck fill is raised by mud wave action, while the rock ks into the mud. The use of rock fill materials is therefore limited. It may, however, be used to displace shallow mud layers by allowing it to sink to firm ground. The types of fill which are commonly available adjacent to mud flats, and which can be utilized for various purposes, as explained later, are as follows:

1. Dredged mud
2. Dredged sand
3. Garbage mixed with sand
4. Imported and compacted fill from excavations
5. A combination of 1 through 4

PURPOSES FOR WHICH VARIOUS TYPES OF FILL CAN BE UTILIZED

Generally speaking, there is some use for practically all types of fill, be it for filling in a cavity in the ground or for raising ground level of large areas. The following enumeration of the various purposes for which fills can be used is applicable to mud flats, submerged land, or generally, soft low lands:

(1) Recreation
(2) Industry and commerce
(3) Residence

Recreation

Development of recreational areas by filling is treated first, largely because this kind of development is less sensitive to poor selection of fill materials. Appreciable future settlement in areas of parks, tennis courts, and sundry light structures with light access roads can be easily accommodated by providing initially for higher ground levels. Differential settlements would have little bearing on such development, except perhaps in the immediate area of light structures, which can receive special local treatment. It is noted that the lighter the fill material, the less consolidation will occur in the underlying mud. Since no concentrated loads are to be imposed on such areas, the fill type most suitable, least costly, and abundantly available is garbage with some admixture of wind blown or dredged sand. The admixture of sand is necessary to fill the large voids in the garbage material, which otherwise would compress and decompose internally. A mixture of 60 percent garbage with 40 percent dredged or windblown sand would constitute a suitable fill material for recreational areas.

This does not preclude the use of superior fill types such as dredged mud, dredged sand and/or imported fills, but as a matter of economy the inferior fills will suffice. Where the area to be filled for this purpose is submerged, it may be advisable to raise it above mean sea level by dredged mud and continue with garbage and sand fill.

Industry and Commerce

Fills placed in areas to be used for industrial and commercial purposes have to be selected and fairly incompressible. By select fill is meant fill which is well-graded with the absence of large voids which might permit the mud to squeeze upward into the fill body. This requirement is mandatory not only because of the building loads that may be imposed upon the fill, but because of roads and utilities. Incompressibility of the fill material, however, will not prevent consolidation in the underlying mud with resulting future settlement which may be large, depending upon the amount of fill placed and the time elapsing between completion of fill and final development.

The final ground surface elevation to which industrial and commercial fills should be raised is controlled by the following factors:

(1) Drainage or flood control including tidal wave
(2) Required thickness of fill for footing loads
(3) Earthquake considerations (not being treated in this report)

The method of placing fills for this purpose must be carefully executed as explained earlier in this report. In the case of submerged land, it is necessary to bring the surface up to or slightly above mean sea level by slow process such as dredging. A cut-off trench filled with sand may have to be employed to prevent mud heave beyond the fill boundary. Tamping of fill material should be avoided until sufficient thickness of fill has been placed to reduce the possibility of punching through the fill into the mud.

The use of heavy fill materials such as decomposed rock is not advisable, at least for use as a fill blanket directly over the mud. Although such materials would be acceptable once in place, the difficulty of placing is almost insurmountable. Heavy materials such as decomposed rock from excavations must be placed by end-dumping, and it is this method that is objectionable and invariably gives rise to the creation of mud waves throughout the fill area. The type of fill most suitable for this purpose is sand, either quarry or dredged. It is pointed out that satisfactory placement of sand can be accomplished by bulldozers and does not require heavy equipment such as sheep's foot and/or heavy rollers, whereas heavy fill material cannot be properly placed without this type of compaction equipment. Once a firm sand blanket has been established, other select fill material such as decomposed rock may be used to bring the fill area up to the desired grade. The thickness of the sand blanket should not be less than three feet.

Residence

The construction methods for residential fills are similar to those used for industrial or commercial fill developments. However, since the avoidance of differential settlement in residential areas is of utmost importance, with regard to streets, utilities, and also building foundations, stricter control of fill quality and method of placing is necessary. Such control is also necessary for the prediction of future behavior of developments. It is impracticable to support residential structures on piles while streets and adjacent areas are allowed to subside. Foundation support for residential buildings should therefore be taken upon the settling fill. Residential developments on fills placed upon bay mud are the most exacting of all the possible uses. Any miscalculation, faulty placement, or undetected condition may result in rapid deterioration of residential subdivisions beyond practical repair.

For the specific purpose of developing mud flats for residential subdivisions, the following requirements are deemed necessary.

(a) The first fill blanket must consist of well-grade granular material, carefully placed. The method placement must be such that at no time is the mu surface distorted or broken with resulting mud wave

(b) Compaction by heavy equipment of the fir three-foot fill blanket should not be allowed. The onl material that can be thus placed without future shrinkage of the fill material itself is either windblown san or dredged sand. Both can be hydraulically placed.

(c) The rate of filling should not be in excess three feet a year.

(d) Ultimate elevations and slopes should be adequate to provide drainage, flood control and safet against tidal waves.

(e) Settlement markers should be carefully place and read at regular intervals in order to establish settlement pattern.

(f) No construction should start prior to the laps of five years after the fill has been completed.

Mat Floating Foundations

Where deep excavations are to be made, it may b feasible to take advantage of the relief load caused b excavation. Substituting foundation loads of high ris buildings for the weight of soil thus excavated is generally referred to as a "floating" foundation. If th weight of the building is exactly equal to the weigh of soil excavated, inclusive of its natural water content, the stresses at the level of the building foundation will be the same as those in nature. If the weigh of the building is less, and the ground water table high, the stresses will be less than those in the natur soil, and a condition may even be created whereb the building may tend to pop out of the ground. "floating" foundation is usually accomplished by providing a thick, reinforced concrete slab (mat) at th bottom of the excavation, and the building column are supported directly on the mat. The periphery wal that then extend from the mat to the ground surfac constitute the basement enclosures. The factors tha will determine whether any stress (upward or downward) exists between the bottom of the mat and th soil, will depend on: (a) the weight of the buildin and/or the weight per square foot of the building the foundation level; (b) the depth of excavation; an (c) ground water table. Provisions to prevent th building from "popping" are customarily made by extending the mat beyond the basement periphery. I the case of the building weight being greater than th uplift forces of the water, settlement problems cause by the semi-infinitely loaded area may be critical; th is particularly true since the foundation mat is practically rigid and cannot adjust to differential settlements other than by tilting. Other problems must b carefully considered in the design of such a foundation, some of which are:

(1) Dewatering the building site.
(2) Stabilization of adjacent areas during construction.
(3) Prevention of heave in the excavation botton
(4) Careful balancing of foundation loads with geometrical configuration of the building to prevent eccentricity of loading.
(5) Provision against tilting.

No mat for a major building is known to have been constructed in the soft bay mud. It is, however, conceivable that a mat foundation for a large building with several stories of basement, could penetrate the upper mud and be founded on the sandy soils directly underlying it.

The effects of earthquakes upon this type of foundation, as compared to the more conventional types, need further study and research. It is noted here that the embedment of the mat in the underlying sand gives rise to at least three different vibration systems: the sand, the mud, and the building.

FOUNDATIONS FOR STRUCTURES LOCATED ON BAY FILLS

Foundation support for any structure located in an area which is subject to subsidence must be established on the basis of the expected utility and performance of the structure. Almost all major structures are sensitive to differential settlements which affect the design strength as well as stability of the structure, and may also eat away the built-in safety factor which is to safeguard against periodic overstresses such as would occur during an earthquake. Minor structures such as residences are less sensitive to differential settlements, but may lose much of their property value due to ugly cracking that results from differential settlements.

From what has preceded, it must be clear that uniform settlement of freshly placed fills on bay mud cannot be expected under ordinary field conditions, nor can the amount of differential settlement be computed on a rational basis unless field conditions of the underlying mud and the manner in which fills are placed are ideal. Therefore, allowance must be made for differential settlements of a detrimental dimension n the case of freshly placed fills. In the case of "well-easoned" fills the amount of differential settlement can e determined from field observations, and design eatures may be incorporated in the building to proide for such settlements.

It is therefore deemed necessary to abstain from any onsideration of building foundations on newly placed ay fills on the ground that there are too many indeerminate factors involved in the prediction of the uture behavior of such fills. With regard to "well-easoned" fills that have shown a certain pattern of ettlement and differential settlement, the assumption an be made that future settlements after construction f buildings will be slow and not in excess of six nches, nor differential settlements in excess of three nches in the following 20 years.

With these assumptions one may proceed to proide foundation support for the buildings themselves n such a manner that any additional settlements due olely to the weight of the building will be negligible. ecommendations for practical and sound foundaions for various types of structures cannot be made s a blanket coverage; a foundation or soil mechanics ngineer should be consulted for each individual strucure. Some obvious problems involving foundation esign are being outlined for guide purposes only.

Residential Houses

A conventional spread footing foundation with nominal embedment is usually satisfactory. Provisions for shimming up differential vertical deflections (such as by installation of jacks) should be provided under all footings.

Industrial and Recreational Buildings

These buildings are usually of large dimension and, although one or two stories in height, may be called upon to carry heavy floor loads. For two-story buildings with heavy upper floor loads, piles may be necessary, although a well-designed grid-footing foundation with nominal embedments may suffice. Concrete slabs on ground may be used, provided they are separated from the foundation footings. Limitation of integral slab dimensions is advisable.

Piled Foundations

There are many reasons or preferences for supporting some major buildings on piles. The reasons may be purely imperative due to nonbearing soils underlying the site at shallow depths, while the preferences may have to do with economy or restrictions on allowable total or differential settlements. The state of the art of pile installation is in the process of developing into a science. In the meantime, there have been a number of casualties in the form of "sour" buildings that have contributed to the scientific "post mortems" of why piled foundations went sour. The briefest way to outline the deficiencies of some pile foundations is to enumerate the necessary information required for a good foundation design. In sequence of importance are these factors:

1. Thorough test boring program and soils engineering evaluation of soil characteristics.
2. Ascertainment of satisfactory soil strata within the bearing length of the piles as well as below the tip elevation of the piles.
3. Consideration of downdrag on the pile due to fills overlying compressible soils.
4. Engineering supervision by soils engineers for every pile installed.

The case history of the Oakland Auditorium indicates that, although the pile foundation was constructed in accord with the practice of the time, none of the above factors were given attention. There have been more dramatic cases of pile failures which were attributable merely to a single factor. Although all pile design and installation have similar engineering requirements regardless of location, yet where piles are driven through mud into superior soil strata, the significance of all the four factors stated above becomes more obvious.

The question of the ability or inability of piles driven through mud to withstand earthquake forces has been debated in the profession for many years. Although mud is a semi-fluid mass, it offers considerable resistance to impact loads, much the same as does water. Moreover, since the mud itself is subjected to vibrations, the embedded pile tends to vibrate with it. At times of opposite motion the mud tends to flow around the pile, but no failure of the pile is likely to occur as long as the tip of the pile is free to rotate. In the case of piles driven through the overlying mud and into the underlying rock, the problem may be critical. The vibrating mud and that section of the pile within the mud may cause the pile to shear off above the rock surface, which vibrates in an absolutely different manner from the mud and pile.

Experience with pile foundations in mud areas following the earthquake of 1906 shows that quite a number of buildings on pile foundations survived the quake, but it must be emphasized that none of those piles was embedded in the underlying rock. A recent trend to embed piles in the rock has been noticed, but it will have to be seen how these piles behave during a heavy earthquake, before final conclusion can be reached.

CASE HISTORY 1.
SAN FRANCISCO WATERFRONT

By CHARLES H. LEE, Consulting Engineer

(Figures follow Page 58)

The San Francisco waterfront, including the lower Market Street area below Montgomery Street from Telegraph Hill (Lombard Street) to Rincon Point (Harrison Street), is on filled ground placed in the former Yerba Buena Cove. The area of the fill from the original bay shoreline of 1849, which skirts First and Montgomery Streets, to the existing sea wall is 256 acres; the maximum width 1,200 feet and the length 7,000 feet (figure 1).

Yerba Buena Cove, before filling, was a crescent-shaped indentation in the original shoreline and in early days was the anchorage area for most of the shipping. The original community of Yerba Buena was adjacent to the western end of the cove and surrounded Portsmouth Square near the present intersection of Columbus Avenue and Montgomery Street. At low tide a broad mud flat was exposed in the northern portion of the cove opposite the built-up area, but in the southern portion the shore was sandy.

San Francisco waterfront is underlain by 10 to 100 feet of soft mud, increasing in depth toward San Francisco Bay (figure 2). Below the mud are alluvial beds of silty and clayey sands and sandy firm clay, interbedded with stiff marine clays. Bedrock lies at depths varying from -20 feet near the hills to -275 feet MSL under Mission Street. Bedrock depths are greatest in the Fulton-Mission Street area which overlies a deep Franciscan rock ravine or canyon, with stream channel, which, at an earlier geological age, gathered surface run-off from the Hayes and Noe Valley drainage areas and delivered it to Sacramento-San Joaquin River, west of Alcatraz Island (figure 3).

History of Fill

Early in the history of the community the need of wharfage to accommodate unloading of ships was apparent.[1] The first wharf was constructed in 1847 at the corner of Broadway and Battery Streets. Others followed rapidly, and by 1850 total wharfage exceedec 6,000 feet in length, extending from 500 to 900 fee into the bay along parallel streets. By 1854 the tota length had increased to two miles including cros streets supported by piles.

The first fill was placed at Laguna Salada, a pear-shaped salt water inlet at Jackson and Montgomery Street which extended almost to Kearny Street (figur 1). The Montgomery Street bridge had been constructed across the outlet in 1844, but in 1849 it wa replaced with a fill which covered the whole lagoon Filling of the cove was accomplished by dumpin from wharves and cross-streets, forming dykes whic enclosed square ponds of stagnant water containin refuse and hulls of old ships.

Active filling of "water lots" commenced in 185 and continued until 1873. The first "water lot" to b filled was on California Street west of Battery. Th fill material consisted of dune sand with an admixtur of rubbish, building rubble, old ship hulls, and studde with a great number of wooden piles which originall supported wharves constructed while the area was stil the open water of Yerba Buena Cove. Much of the fil material was excavated in the Market Street area. Larg sand dunes existed there, one between Second an Third Streets exceeding a height of 80 feet (figure 2 and another at the corner of Grant Avenue reaching height of 89 feet above MSL. Short haul transportatio was by horse and cart, long haul by steamshovel an small railway cars moving as much as 1500 cubic yard per day. A "steam paddy" imported by steamer wa working constantly from 1852 to 1854 and from 185 until 1873 loading into small railway cars and dumpin into the water lot areas. It has been estimated that a much as 22 million cubic yards was thus placed.[2] San fill was dumped directly into the water without compaction. The thickness of fill as finally placed varie from 25 feet to 30 feet along and to the north o Market Street, but south of Market was as great a 40 feet (figures 2 and 5).

Sea Wall Construction

The insecurity of building foundations placed directly upon the sand fill was soon apparent as evidenced by differential settlement with cracking, tipping and breaking up of buildings. Erosion fro storm runoff was serious, and in addition, tidal wate were continually eating away, washing loose sand ou into the deeper waters of the bay.

In "The Annals of San Francisco"[3] Frank Soul pointed out the absolute necessity of a great sea wa along the outer front of the harbor to protect th "beach and water lots." He also recommended dee and closely spaced piling for support of larger buildings in this area. In the Report of the City Surveyo of San Francisco, dated April 7, 1856, a sea wall co structed from Rincon Point to the eastern base c Telegraph Hill was declared to be a work "immed ate and paramount to every other consideration." Th State Board of State Harbor Commissioners was cr ted April 24, 1863 for administration of the Harbor Front and by 1869 the sections of wall between Mission and Pacific Streets and between Vallejo and Union Streets were completed. The alignment was angular following street lines along the then "City Front", but it was soon discovered that great quantities of "silt" accumulated in the angular recesses. This was thought to have been caused by trapping of sediment discharged into the bay and carried along the waterfront by tidal currents. It is possible, however, that the source of the so-called "silt" was bottom heave of the underlying mud produced by the weight of the rock fill in the sea wall (figure 2). It was decided to remedy this situation by selecting a new line beyond the existing one, conforming to bay currents and lying in water from 10 to 24 feet in depth. Construction commenced in 1878 and by 1916 it was completed from the east side of Taylor Street to southern county line. The alignment was essentially that of the present sea wall.

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[1] Historic data regarding Yerba Buena Cove fill and the sea wall have been obtained largely from the report of the Subsoil Committee, San Francisco Section, American Society of Civil Engineers entitled "Subsidence and the Foundation Problem in San Francisco" September 1932, supplemented by research at the Bancroft Library at University of California, and elsewhere.

[2] "History of California" by Hubert H. Bancroft, Vol. VI, page 200.

[3] "The Annals of San Francisco" by Frank Soulé, John H. Gihon and Jam Hisbet, 1854, page 529.

The original sea wall consisted of quarry-run rock obtained largely from the north and east slopes of Telegraph Hill, and dumped "pell-mell" along the center line, with larger rock on the water side and brought to a width of 13 feet at mean low tide. Prior to constructing the rock embankment a trench was excavated 40 feet wide at the bottom, 20 feet below mean low tide (-22 feet MSL). The fill was allowed to settle for several months until a "firm foundation" was attained and then topped with a concrete wall supported by wooden piles. All contracts were completed by 1869. During inspection in 1871, the Harbor Engineer found the sea wall to be in perfect condition with no evidence of lateral movement, and settlement of only one to three inches, considered to be due to compacting of the rock fill.

Plans for the present sea wall were prepared in 1878 based upon rather complete preliminary boring information. Along the Embarcadero from the foot of Powell to Pacific Street "soft mud" was found varying in depth from 4 to a maximum of 15 feet, below which was encountered "stiff mud." (After construction it was found that the rock fill had penetrated into the soft mud at different points from 4.9 feet to a maximum of 10.3 feet).

Between Pacific Street and Harrison Street the depth of soft mud was much greater. Recent borings made for the Rapid Transit District encountered it to a depth of —117 feet below MSL at the head of Pier No. 4 near the foot of Market Street.[4] Similar borings penetrating the present sea wall at the same location show that the rock fill has sunk 20 feet into the original mud bottom of 1852 (figure 2).

Beyond Harrison Street and skirting Rincon Hill the mud was found to be shallower, but near the foot of Channel Street at Piers 48 and 40 in the China Basin area another submerged valley was found partially filled with very soft mud. A similar bedrock valley underlies the Islais Creek Basin area where recent borings[5] have encountered soft bay mud to a depth of —60 feet MSL.

Prior to construction of the present sea wall a trench 20 to 30 feet wide at the bottom was excavated in the soft mud to a depth varying from 20 to 35 feet below mean low water. The loose broken-rock embankment was placed in this trench and the natural slope of the material under water was carried back to low water level on the water side and City Base on the shore side. The width of the embankment, at the level 20 feet below mean low tide, was 100 feet. A layer 4 feet in thickness of hand-laid large rock was placed on the water slope for protection against wave action. The area behind the wall was filled with defective or soft stone, quarry scrapings, dune sand, and mud dredged up from the waterfront. After settlement had taken place the area behind the sea wall was paved to form the Embarcadero.

Later, after construction, it was found that with the ebb and flow of the tide, fill material from behind the wall seeped through and caused settlement of the paving, particularly in the area between Mission and Brannan Street. To correct this the original design was modified by construction of a pile-supported concrete wall at the center of the fill with base 11.5 feet wide at elevation —30 feet San Francisco City Datum and top at street level.[6]

Settlement of Sea Wall

Settlement rate at the present sea wall south of Harrison and north of Vallejo Street has varied from 0.02 to 0.03 feet per year except in the piled area at the Ferry Building. A section of sea wall between Harrison and Howard Streets, originally brought to City Base, settled one foot over a weekend with lateral movement of 1.5 foot toward the bay. Settlement continued, and at the foot of Folsom Street by 1917 amounted to a maximum of 4.5 feet with 2 feet of lateral movement. The top elevation was restored to 0.5 foot below City Base in 1917 but settlement continued at the rate of 0.2 to 0.3 foot annually, and by 1930 the wall had settled approximately 4 feet from Folsom Street to beyond Howard Street. This section was restored to grade elevation in 1932. Several buildings in this area were also damaged. The old Post Office Building facing Mission Street between Spear and Stewart Streets subsided badly and unevenly and in 1923 portions of the buildings had to be raised to original grade.

Settlement of Fill

In addition to settlement along the sea wall, there has been settlement of the main body of the sand fill back of the sea wall. The most spectacular subsidence occurred along Market Street between First Street and the Embarcadero at the time of the earthquake in 1906 (photo 1). A maximum settlement of 3 feet occurred at Spear Street and almost 2 feet at Beale Street. The street was brought up to grade after the earthquake, but slow settlement continued amounting by 1923 to from 0.5 foot at Main Street to 1.5 feet at the Embarcadero. This section of Market Street was restored to official grade in 1923, but settlement has continued and by 1955 was 1.8 feet below grade at the foot of Sacramento Street (figure 4).

Rapid Transit, Soil Investigations S-702 San Francisco-Market. Dames & Moore, Plate 1A.

Port of San Francisco, Report on west addition Islais Creek grain elevator. Lee & Praszker, Aug., 1962.

[6] San Francisco City Datum is 8.69 feet above mean sea level.

Between 1925 and 1943 the City Engineer made records at two-to-three-year intervals of settlement at all street intersections located in known areas of subsidence in San Francisco. Detailed study has been made by this office of these records in the San Francisco waterfront area for the purposes of this report. It was found that settlement was most rapid in a strip two to four blocks wide north of Market Street between Embarcadero and Montgomery Street (figure 1). The 14-year (1923 to 1936) rate of settlement in this area varied from .09 foot per year at the Embarcadero to .02 foot per year at Montgomery Street, the accumulated total settlements at these respective locations being 1.3 feet and 0.3 foot. In other portions of the fill the total settlement graded off to 0.1 foot near the margins along the mean high water line of 1849. At many points there was a marked decrease in rate of settlement after the year 1936 as indicated by the fact that the average rate of settlement at 62 street intersections from 1923 to 1936 was 0.025 foot per year while from 1936 to 1943 it was only 0.015 foot per year. Later measurements by the City Engineer indicate that during the period 1945 to 1965 the weighted average, including points at which heave occurred was 0.002 foot per year. It is evident from examination of the records that active settlement over much of the fill eased off after 1936.

Total average settlement along Market Street from Montgomery Street to Spear Street from 1864, the average date of filling, to 1964, as shown by Profile B (figure 2), was 9 feet. Total average settlement for the first 100 years as computed theoretically from figures 1 and 2 of the main report is 4.32 feet. The great discrepancy between these two values suggests the possibility that an appreciable quantity of mud has escaped beyond the sea wall by plastic flow into San Francisco Bay.

Plastic Flow of Mud

The possibility that settlement of the surface in the San Francisco Waterfront may have resulted partially from displacement of mud by plastic flow into the bay beyond the sea wall has been a subject of discussion among engineers for many years. Review of available information as of present date indicates the possibility of obtaining an answer to this moot question. Recent engineering reports prepared in connection with the Golden Gateway Redevelopment Project,[7] and San Francisco Bay Area Rapid Transit[8] contain much pertinent data. These data together with those in other reports and those shown on U.S. Coast and Geodetic Survey hydrographic maps of 1852, 1936 and 1958 provide sufficient information for preliminary quantitative studies, such as shown in the accompanying addendum.

An actual instance of plastic flow of mud from under a filled area has recently been investigated by a member of this firm. This occurred at the Anderson & Cristofani Marine Ways, Innes Avenue and Griffith Street, San Francisco, located in the cove on the north shore of Hunters' Point. This boat building establishment has been in operation for 75 years with an unobstructed entrance channel having at least 7 feet depth at low water, and an open water frontage of approximately 500 feet. Within the past few years fill has been placed upon the open water areas both to the east and west. The fill on the east has been extended out into the water 1000 feet and brought up to an elevation of approximately City Datum. On the west, fill composed of contractor's waste, is being placed progressively to an elevation considerably above City Datum. Concurrent with filling, the mud bottom has visibly heaved in the open water in front of the Marine Ways, necessitating frequent dredging to maintain the entrance channel. This condition has given rise to litigation.

Another well known instance of plastic flow of mud is at the Toll Gate Plaza, San Francisco-Oakland Bay Bridge, where large quantities of fill have been placed upon soft bay mud of considerable depth. Plastic flow of mud from under this fill has occurred with extensive heaving of the adjacent bay bottom, particularly on the north side of the fill. In one area the surface of mud which formerly was barely exposed at low water has been raised by heaving to an elevation several feet above mean high water.[9] This area overlies a deep mud-filled valley and the greatest heaving has occurred above the lowest part of the valley where the mud is the thickest.

These recognized instances of plastic flow of mud beyond the margin of fills and corresponding heaving of the adjacent bottom of the bay invite study of the situation at the San Francisco waterfront. Profiles B and E (figures 2 and 3) accompanying Appendix of this report are pertinent to such a study. They have been compiled from engineering reports recently submitted to the San Francisco Rapid Transit Distric and the San Francisco Redevelopment Agency, and from the progressive hydrographic maps of San Francisco Bay issued by the U.S. Coast and Geodetic Survey beginning in 1853. These two profiles lie at righ angles to each other, one along Market Street and it projection into San Francisco Bay, and the other between Beale & Main Streets from Telegraph Hill to Rincon Point. They show the elevation of the surface of the mud at the bottom of the bay as it was in 1852 and the present bottom of the sand fill which wa placed thereon during the succeeding 20 years. From these and other similar profiles the average settlemen has been computed of the surface of the mud unde the weight of the fill throughout the San Francisco Waterfront area during the 100-year period since fil was placed. It was found to be 8.7 feet.[10]

The settlement resulting from consolidation of th mud by squeezing out of water was then computed from the measured present porosity of the mud a different levels, as compared with the porosity at simi lar levels in 1852. The latter values were derived by test of undisturbed samples of mud at different levels presently obtained from a test boring at the pier hea ne at Pier 14 in San Francisco Bay, 800 feet beyond ie sea wall. The average settlement of the mud by onsolidation during the 100-year period as thus comuted was 3.8 feet.[11]

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[7] Report, Foundation Conditions, Golden Gateway, Embarcadero—Lower Market Street Approved Development Project Area E-1, to Redevelopment Agency of the City of San Francisco by Lee and Praszker, Consulting Engineers, January 1960.

[8] Report, Soils Investigation, Contract S-702, San Francisco, California, San Francisco Bay Area Rapid Transit System to Parsons Brinckerhoff-Tudor Bechtel by Dames & Moore, Foundation Engineers, Aug., 1964.

[9] Engineering geology of San Francisco Bay, California by Trask & Ralston Bul. Geol. Soc. of Am. Vol. 62, pp 1079-1109, 1951, Figs. 12 and 13

[10] See Addendum, Appendix 1, for details.

There thus appeared to be an unaccounted for dif-rence of 8.7 − 3.8 = 4.9 feet. Applying this amount the fill of 256 acres within the San Francisco Water-ont area results in a volume of unaccounted for set-ement of 1256 acre feet. There is a possibility that is volume represents mud which has been displaced om under the fill by plastic flow out into the open ıy. There is much to support this proposition.

Prior to the sea wall construction and between 1853 ıd 1859 frequent soundings to the mud were made the open water along existing wharves beyond the ity Front and beyond any sandfilled area.[12] These undings indicated a progressive rise in the mud bot-m beginning in 1853 which by 1859 amounted to as uch as 20 feet at the end of the wharves, and pro-ıced a pronounced "bowing out" of depth contours eyond those of 1853.

Table 1. Record of rising mud bottom—waterfront—
City of San Francisco.
1853 to 1859

		Rise in mud bottom—feet			
		At line of "city front"		At end of wharf	
Wharf	Length in feet	Total	Yearly	Total	Yearly
llejo St.	660	5½	1.0	11	1.8
oadway	470	12	2.0	20	3.3
cific St.	615	8	1.3	17	2.8
kson St.	660	13	2.1	22	3.7
ishington St.	660	5	0.8	20	3.3
ay St.	700	14	2.3	29	4.8
mmercial St.	330	15	2.5	13	2.1
irket St.	770	12	2.0	20	3.3
ion St.	470	13	2.1	10	1.6

Filling of "water lots" within the city commenced 1852 and continued actively into the late 1860's. The served heaving of the bay bottom beyond the front-e limit of the city appears to be the result of this ling, since the fill did not extend beyond the front-e limit, and since an adequate source of silt was not ailable for deposition by tidal currents (figures 1 and . Such heaving could only result from plastic flow mud from beneath the fill toward the deeper un-nfined mud area beneath the bay.

Since construction of the present sea wall, constant edging of the berthing space between piers has been cessary to maintain draft depth for ships at low le. This would indicate a continuation of the heaving ocess since there is no continuing active local source silt available for tidal currents to pick up and carry ng the waterfront.

Examination of the successive hydrographic maps of San Francisco Bay published by the U.S. Coast and Geodetic Survey and its predecessor the U.S. Coast Survey, beginning in 1853, shows that over the years there has been a progressive "bowing out" of depth contours opposite the former Yerba Buena Cove extending at the present time even beyond the 60 feet depth below mean low water (figure 7). In order to evaluate the extent of the rise in the bottom of the bay at various dates, five profiles of the bay bottom have been drawn. Profiles A and C which follow the limiting points of inflection of the "bowing" indicate but minor changes in bottom elevation (figures 8 and 9). Profiles B, F and G which traverse the "bowed out" area (figures 2 and 10) indicate significant rise in bottom elevations extending almost to the 80 foot depth contour. Computation of the volume represented by the rise in the bottom of the bay is summarized in the following table using values scaled from the profiles and applying to the period 1852 to 1938 embracing the period from commencement of filling to end of appreciable settlement.

The resulting volume of 1385 acre feet is closely equivalent to 1250 acre feet, the volume of unaccounted settlement in the fill. If plastic flow has occurred, it is possible that after release from the surcharge of the fill the mud has reabsorbed water and attained a void ratio approaching that of the unconfined soft mud encountered in the test boring at the head of Pier 14. If the above calculated volume of 1385 acre feet be adjusted to the void ratio of the first 15 feet of mud under the fill, the resulting volume is 1110 acre feet. If this volume is assumed to have passed through the vertical mud section under the sea wall of 270,000 square feet (figure 5) during the period 1860 to 1960 the average rate of flow would have been 1.8 feet per year.

It is concluded from these studies that general movement of mud by plastic flow has occurred from beneath the San Francisco waterfront fill and that migrated mud now lies beneath the water of the open bay

Table 2. Volumetric computation of rise in bay bottom
offshore from San Francisco waterfront,
1852 to 1938

Profile	Length (feet)	Average rise of bottom (feet)	Area of vertical section of rise (sq. feet)	Average distance between profiles (feet)	Volume of rise between profiles (thousand cu. feet)
A	3,500	0	0	1,000	13,450
F	4,200	6.4	26,900	980	28,450
B	3,400	9.8	33,300	1,240	29,600
G	2,400	6.0	14,400	1,250	9,010
C	1,800	0	0		
Total volume of rise in cubic feet					60,510
Total volume of rise in acre feet					1,385

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[11] ee Addendum, Appendix 1, for details.

[12] lap of the city front and profiles of wharves showing the progress of filling, from the Report and Condition Requirements of the City of San Francisco by James P. Robinson, 1859. Original map in possession of Bancroft Library at University of California.

beyond the sea wall. The extent to which movement under the sea wall has taken place since 1938 is doubtless much less than previously, although there has probably been readjustment with local movement of accumulated mud under the bay. There is possibility of renewal of general movement following a major earthquake accompanied by important local settlement within the filled area such as occurred in 1906 (photo 1). Such possibility might be the subject of further more detailed investigation.

Computations in Re Plastic Flow of Mud, San Francisco Waterfront

As a basis for these computations pertinent data have been compiled from various available engineering reports. Data applying to subsurface materials underlying Market Street as compiled from Rapid Transit reports are as follows:[13]

Natural Dune Sand:

1. Fine sand above water table, uncompacted (average 15 samples Van Ness Avenue to 8th Street).
   —dry weight 100 pounds cubic foot
   —void ratio 0.63
2. Fine sand below water table, naturally compacted (average 5 samples 8th Street to 4th Street).
   —dry weight 108.2 pounds cubic foot
   —void ratio 0.54

Fill Material:

1. Fine sand below water table, naturally compacted (average 6 samples Beale to Spear Streets).
   —dry weight 108.2 pounds cubic foot
   —void ratio 0.54.

Soft Bay Mud:

1. Specific gravity (average 12 samples Head Pier No. 14 to Stewart Street) 2.62.
2. Unit dry weight and void ratio.

No fill or surcharge (Test Boring S-702-1 at Head Pier No. 14).

		Unit Dry Weight	
Depth Feet	*Number of Samples*	*Pounds Cubic Foot*	*Void Ratio*
0– 15	2	50	2.27
15– 30	interpolated	65	1.52
30– 50	4	72.5	1.31
50– 70	7	74.5	1.24
70–100	11	76	1.19

Fill or surcharge, 25 to 30 feet of sand after 100 years.

(Test Borings S-702-3, S-702-4, S-702-5, S-702-6, S-702-7).

		Unit Dry Weight	
Depth Feet	*Number of Samples*	*Pounds Cubic Foot*	*Void Ratio*
16– 30	3	64	1.61
30– 50	16	72	1.32
50– 70	10	74.5	1.24
70–100	7	74.5	1.24

Per cent consolidation of mud underlying Market Street.

Depth Feet		*Consolidation %*
0– 15		
16– 30	1 — 2.61/3.27 =	20
30– 50	1 — 2.32/2.52 =	8
50– 70	1 — 2.24/2.31 =	3
70–100 [a]	1 — 2.24/2.24 =	0

[a] Heave area adjacent to sea wall (figure 2).

Total consolidation of mud underlying Market Street.

Depth Feet		*Weighted Averag Settlement Feet*
0–15	15′ × .20 × 1 =	3
15–30	15′ × .08 × 0.60 =	0.72
30–50	20′ × .03 × 0.20 =	0.12
50–70	20′ × 0 × 0.20 =	0
	TOTAL	3.84

Similar data for unit weight and void ratio applyin to the fill underlying *Golden Gateway Project*[14] ar as follows:

With fill or surcharge 20 to 35 feet of sand, 10 years. (Test Borings 1, 2, 3, 4, 5, 6 and 7).

		Unit Dry Weight	
Depth Feet	*Number of Samples*	*Pounds Cubic Foot*	*Void Ratio*
16– 30	5	61	1.67
30– 50	7	68.8	1.37
50– 70	8	70.3	1.33
70–100	11	73.7	1.22

Per cent consolidation of mud underlying Golden Gateway Project.

Assumes void ratios of mud with zero fill same as f Market Street. (Test Boring S-702-10)

Depth Feet		*Consolidation %*
0– 15		
16– 30	1 — 2.67/3.27 =	19
30– 50	1 — 2.37/2.52 =	6
50– 70	1 — 2.33/2.31 =	0
70–100	1 — 2.22/2.24 =	0

Total consolidation of sand underlying Golden Gateway Project.

Depth Feet		*Weighted Avera Settlement Fee*
0–15	15′ × .19 =	2.8
15–30	15′ × .06 =	0.9
30–50	20′ × 0 =	0
	TOTAL	3.7

From the above data has been computed the tot decrease in volume of the mud underlying the S Francisco Waterfront as caused by squeezing out water. These computations are summarized as follow

---

[13] Report, Soils Investigation, Contract S-702, San Francisco, California, San Francisco Bay Rapid Transit System to Parsons Brinckerhoff-Tudor Bechtel by Dames & Moore, Foundation Engineers, Aug., 1964.

[14] Report, Foundation Conditions, Golden Gateway, Embarcadero-Lo Market Street Approved Development Project Area E-1, to Redevel ment Agency of the City of San Francisco by Lee and Praszker, C sulting Engineers, January 1960.

[15] Using value of specific gravity for mud as 2.62, instead of 2.70 used Golden Gateway report.

Table 3. Volumetric computation of consolidation in feet depth, San Francisco waterfront, 1852 to 1936

hickness of mud —feet	Market Street		Golden Gateway project	
	Consolidation		Consolidation	
	Percent	Feet-depth	Percent	Feet-depth
15	20	3.0	19	2.8
-30	8	0.72	6	0.9
-50	3	0.12	0	0
-70	0	0	0	0
-100	0	0	0	0
Total		3.84		3.7

OTE: Thickness of sand fill is 25 to 30 feet; depth to water table is 8 feet along Embarcadero to approximately 13 feet along First and Battery Streets.

Assuming that 3.8 feet, the average of these two lues, is representative of the whole filled area, the crease in volume within the 245 acre fill area due consolidation has been computed to be 965 acre et. This consolidation can be considered as having curred during the 100-year period 1860 to 1960.

Total settlement of the San Francisco waterfront curring since 1852 can be obtained by comparison of e profile elevations of the bottom of Yerba Buena ve in 1852 before filling,[16] and profile elevations of e bottom of sand fill at the present time. The latter e available at four locations, one along Market Street,[17] one near the Embarcadero at right angles to Market Street,[18] one from Broadway to Mission Street between Main and Beale Streets at right angles to Market Street,[19] and one from Kearny near California Street to Embarcadero parallel to Market Street.[20] These profiles have been reproduced herein as figures 2, 3, 5 and 8, showing both original bottom in 1852, the present bottom of the sand fill, and present street grades.

The average settlement of the sandfill indicated by each of these profiles has been taken from them with the following results:

Kearny Street to Embarcadero at Broadway (Profile A)	4.8 ft.
Market Street (Montgomery to Embarcadero) (Profile B)	9.0 ft.
Embarcadero (Vallejo to Harrison) (Profile D)	9.7 ft.
Beale and Main Sts. projected to Broadway (Profile E)	10.0 ft.
weighted average total settlement	8.7 ft.

It is to be noted that this quantity much exceeds the settlement by consolidation previously computed to be 3.8 feet, the unaccounted for difference amounting to: 8.7 feet − 3.8 feet = 4.9 feet. This amount applied to the fill area of 256 acres gives a volume of unaccounted for settlement of 1250 acre feet.

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[16] .S. Coast Survey, City of San Francisco and vicinity, California, issued 1853.

[17] Plate 1 A, Rapid Transit Soil Investigation, S-702, San Francisco—Market, by Dames and Moore, 1964.

[18] Diagram 12, Redevelopment Golden Gateway Foundation Conditions, by Lee & Praszker, Consulting Foundation Engineers, 1960.

[19] Diagram 14, same report.

[20] Diagrams 12 and 14, same report.

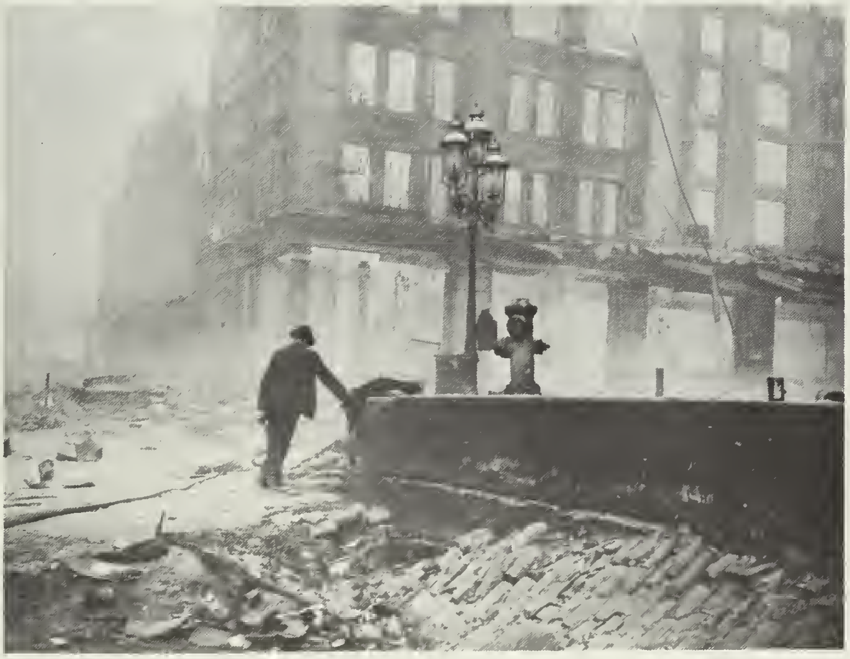


Photo 1. Sunken street level—corner Spear and Market Streets, San Francisco. April 27, 1906.

ILLUSTRATIONS

Page

59 Figure 1. Subsidence contours 1923–1936, San Francisco waterfront.

60 Figure 2. Subsurface profile B. Market Street and projection into San Francisco Bay.

61 Figure 3. Subsurface profile E. Broadway to Market Street between Beale and Main Streets.

62 Figure 4. Profile of south side of Market Street showing subsidence after earthquake of 1906 and from 1906 to 1923 and after restoration of grade from 1923 to 1955.

63 Figure 5. Subsurface profile D. Foot of Market Street adjacent to Embarcadero from Telegraph Hill to Rincon Point.

64 Figure 6. Map of the City Front and profile of wharves showing the progress of filling 1853 to 1859.

65 Figure 7. Map of bottom of San Francisco Bay adjacent to San Francisco waterfront showing location of profiles A to G and depth contours 1852, 1938, and 1958.

66 Figure 8. Subsurface profile A. Kearney Street near California to Embarcadero through Broadway.

67 Figure 9. Subsurface profile C. Folsom Street with extension into San Francisco Bay.

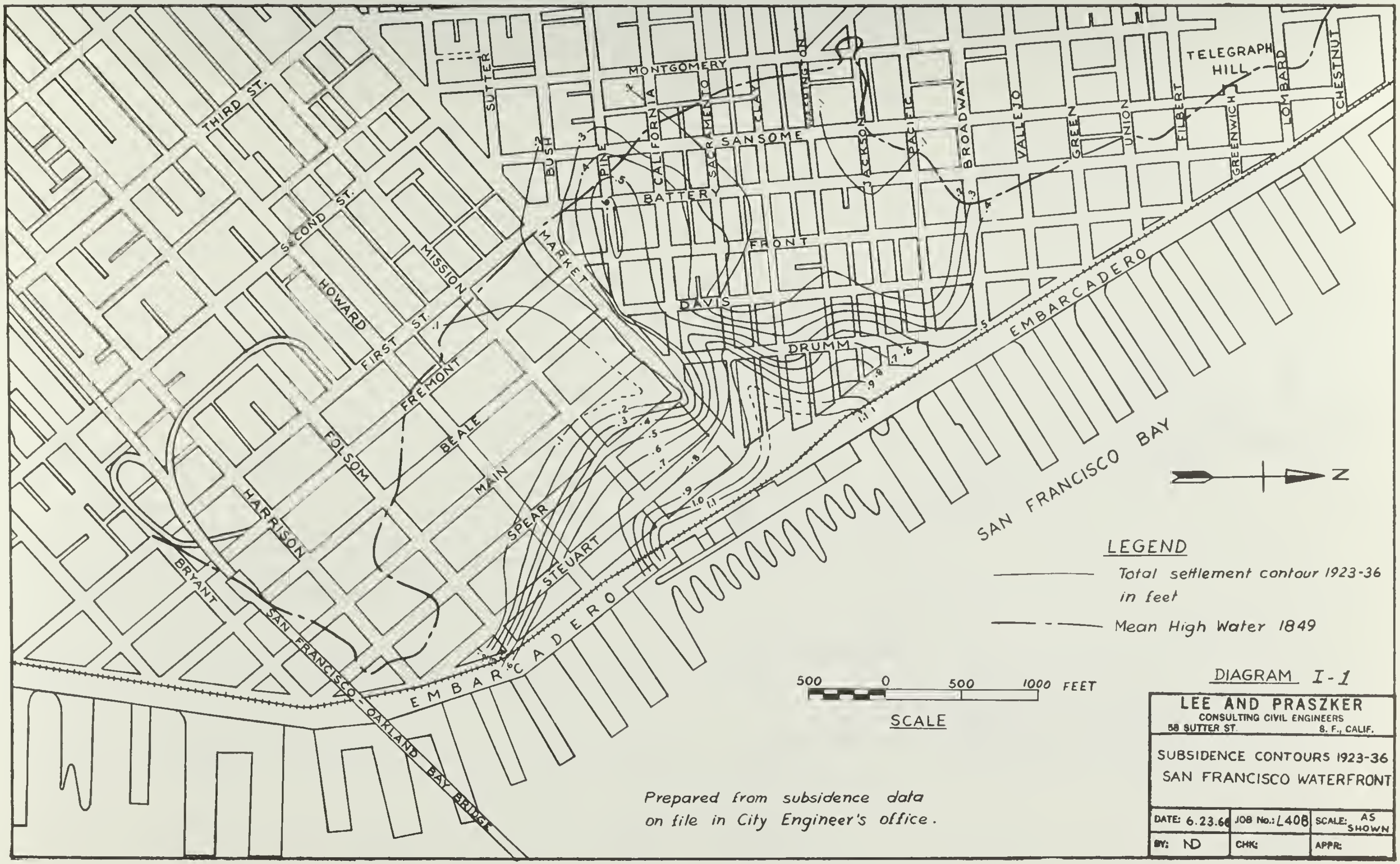
68 Figure 10. Submarine profiles F and G, showing depth contours bottom of San Francisco Bay adjacent to former Yerba Buena cove 1852, 1938, and 1958.

57 Photo 1. Sunken street level—corner Spear and Market Streets, San Francisco. April 27, 1906.

55 Table 1. Record of rising mud bottom—waterfront—City of San Francisco.

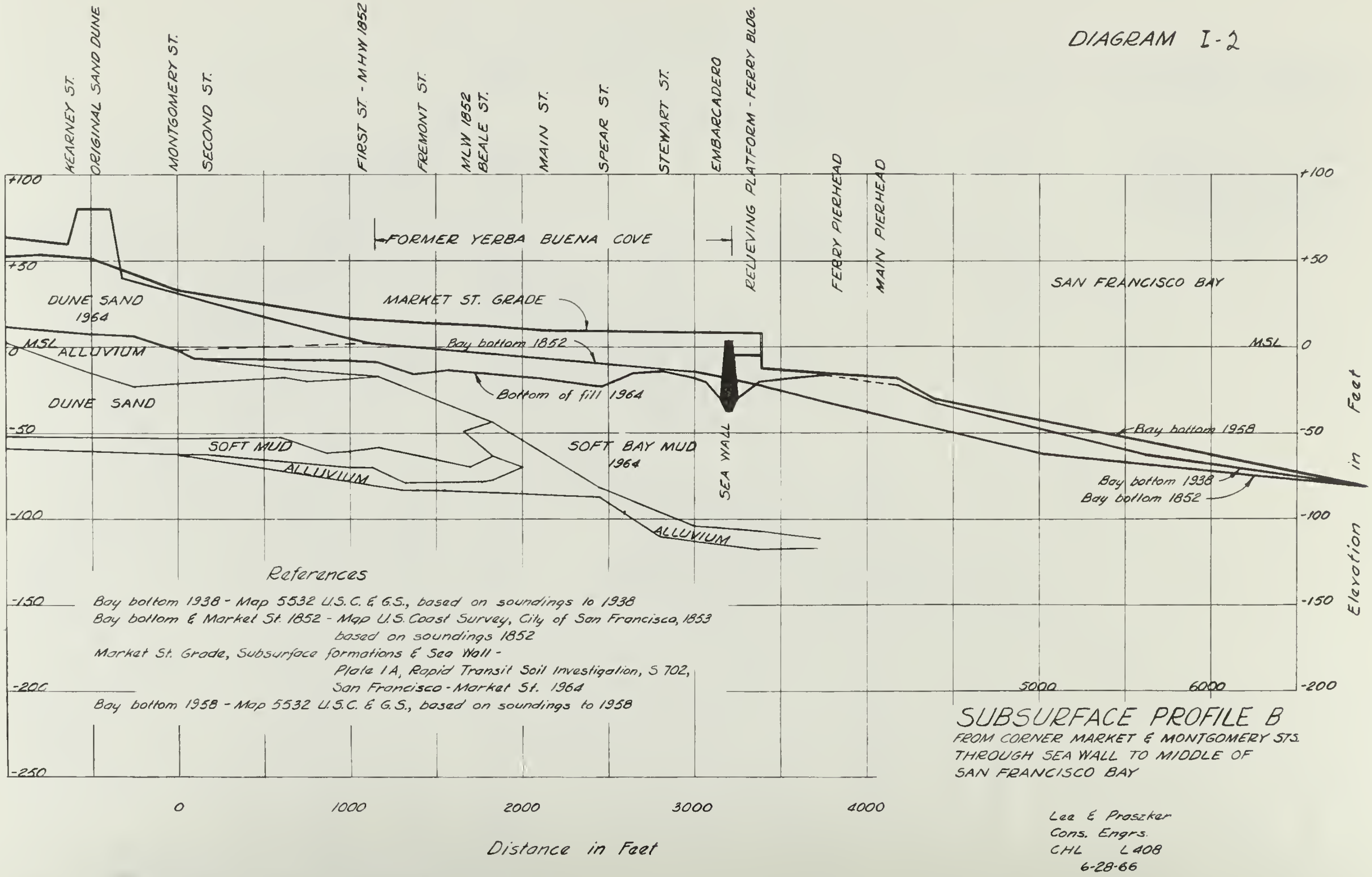
55 Table 2. Volumetric computation of rise in bay bottom offshore from San Francisco waterfront, 1852 to 1938.

57 Table 3. Volumetric computation of consolidation in feet depth, San Francisco waterfront, 1852 to 1936.



Prepared from subsidence data on file in City Engineer's office.

Figure 1. Subsidence contours 1923–1936, San Francisco waterfront.



References

Bay bottom 1938 - Map 5532 U.S.C. & G.S., based on soundings to 1938
Bay bottom & Market St. 1852 - Map U.S. Coast Survey, City of San Francisco, 1853 based on soundings 1852
Market St. Grade, Subsurface formations & Sea Wall - Plate 1A, Rapid Transit Soil Investigation, S 702, San Francisco - Market St. 1964
Bay bottom 1958 - Map 5532 U.S.C. & G.S., based on soundings to 1958

Figure 2. Subsurface profile B. Market Street and projection into San Francisco Bay.

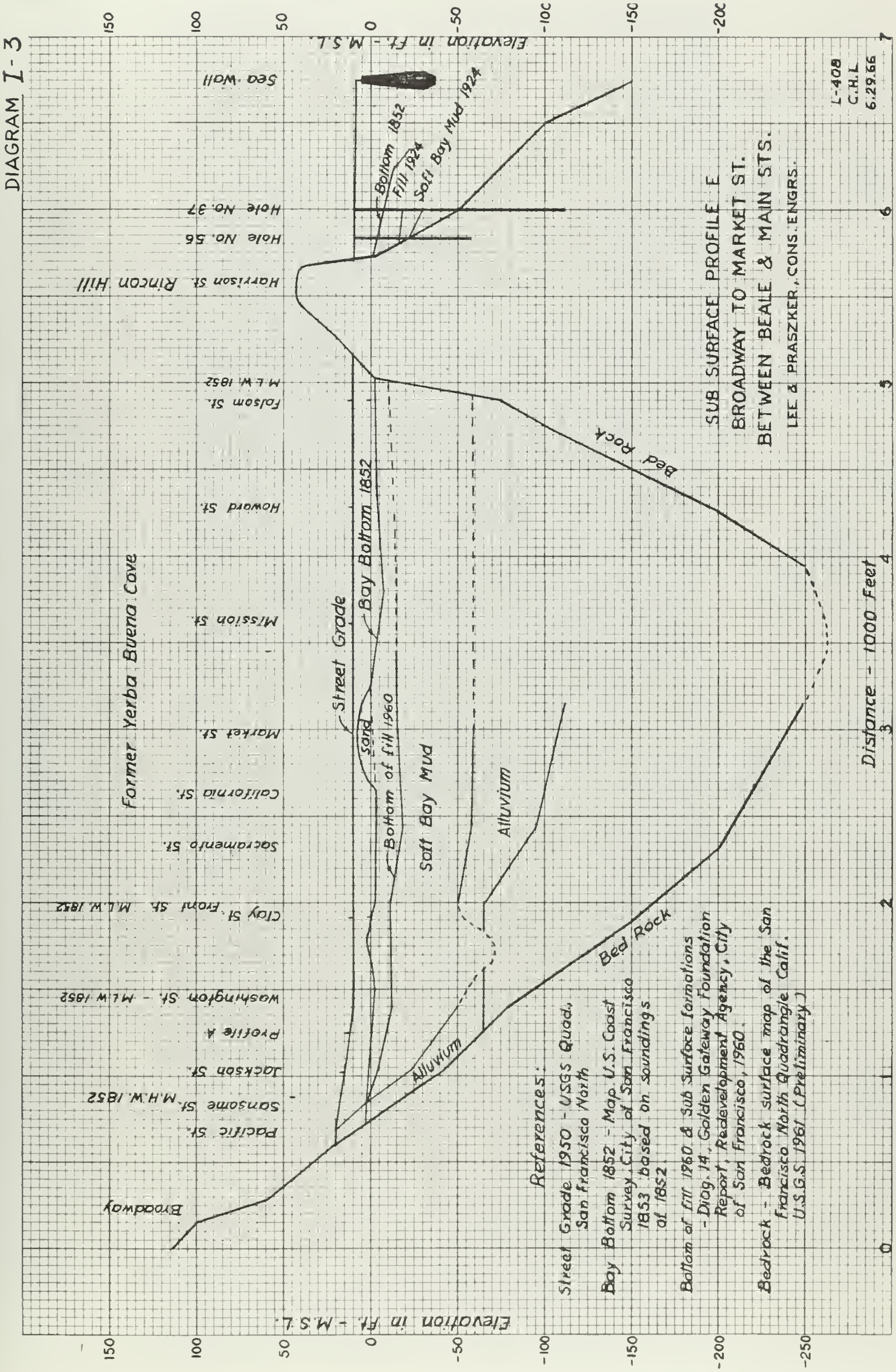


DIAGRAM I-3
SUB SURFACE PROFILE E
BROADWAY TO MARKET ST.
BETWEEN BEALE & MAIN STS.
LEE & PRASZKER, CONS. ENGRS.
L-408
C.H.L
6.29.66
Elevation in Ft. - M.S.L.
150
100
50
0
-50
-100
-150
-200
-250
Distance - 1000 Feet
0
1
2
3
4
5
6
7
Sea Wall
Hole No. 37
Hole No. 56
Harrison St. Rincon Hill
M.L.W. 1852
Folsom St.
Howard St.
Mission St.
Market St.
California St.
Sacramento St.
Clay St. Front St. M.L.W. 1852
Washington St. - M.L.W. 1852
Profile A
Jackson St.
Sansome St. M.H.W. 1852
Pacific St.
Broadway
Former Yerba Buena Cove
Bottom 1852
Fill 1924
Soft Bay Mud 1924
Street Grade
Bay Bottom 1852
Sand
Bottom of fill 1960
Soft Bay Mud
Alluvium
Bed Rock
Alluvium
Bed Rock
References:
Street Grade 1950 - USGS Quad., San Francisco North
Bay Bottom 1852 - Map, U.S. Coast Survey, City of San Francisco 1853 based on soundings of 1852
Bottom of fill 1960 & Sub Surface formations - Diag. 14, Golden Gateway Foundation Report, Redevelopment Agency, City of San Francisco, 1960
Bedrock - Bedrock surface map of the San Francisco North Quadrangle Calif. U.S.G.S. 1961 (Preliminary)

Figure 3. Subsurface profile E. Broadway to Market Street between Beale and Main Streets.

DIAGRAM I-4

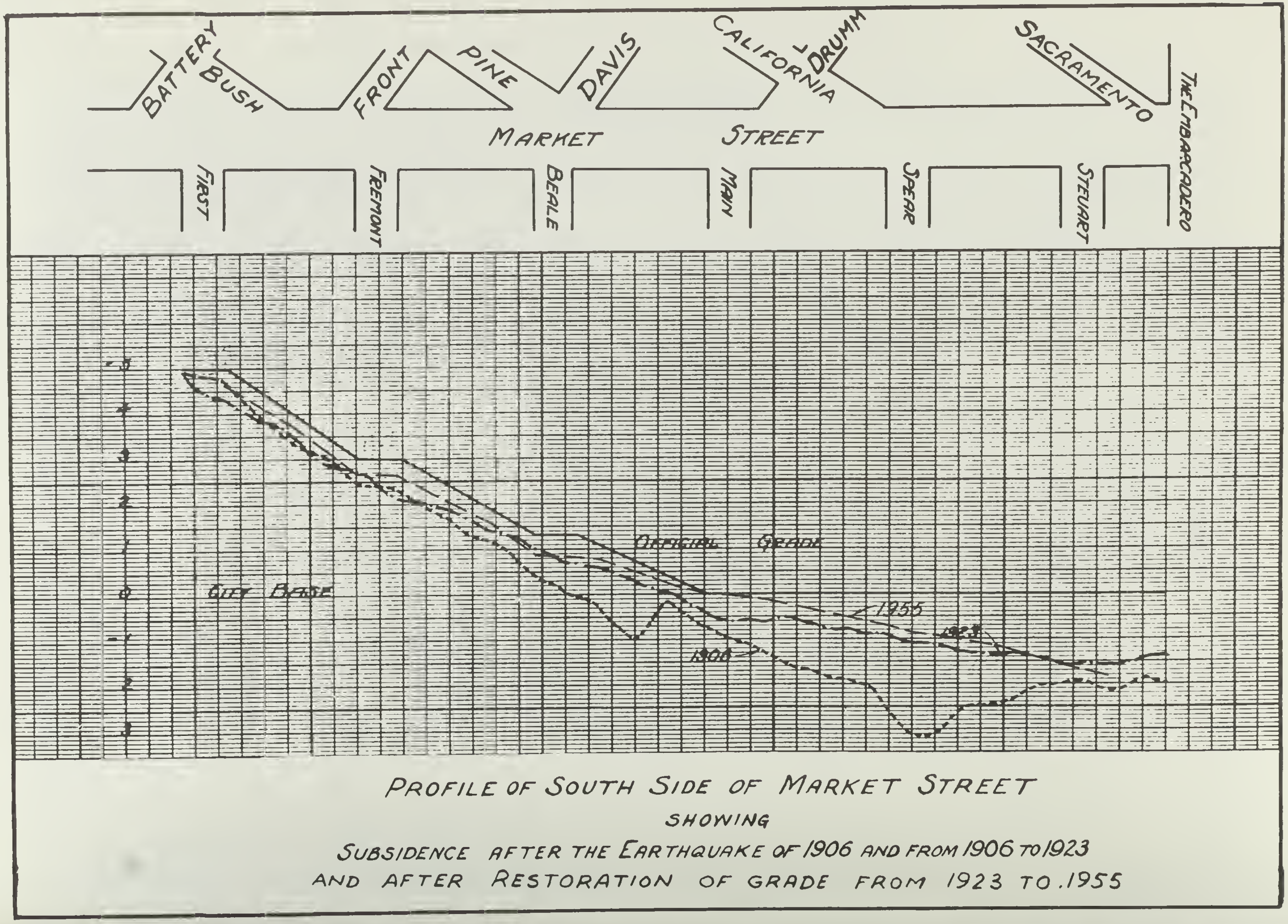


Figure 4. Profile of south side of Market Street showing subsidence after earthquake of 1906 and from 1906 to 1923 and after restoration of grade from 1923 to 1955.

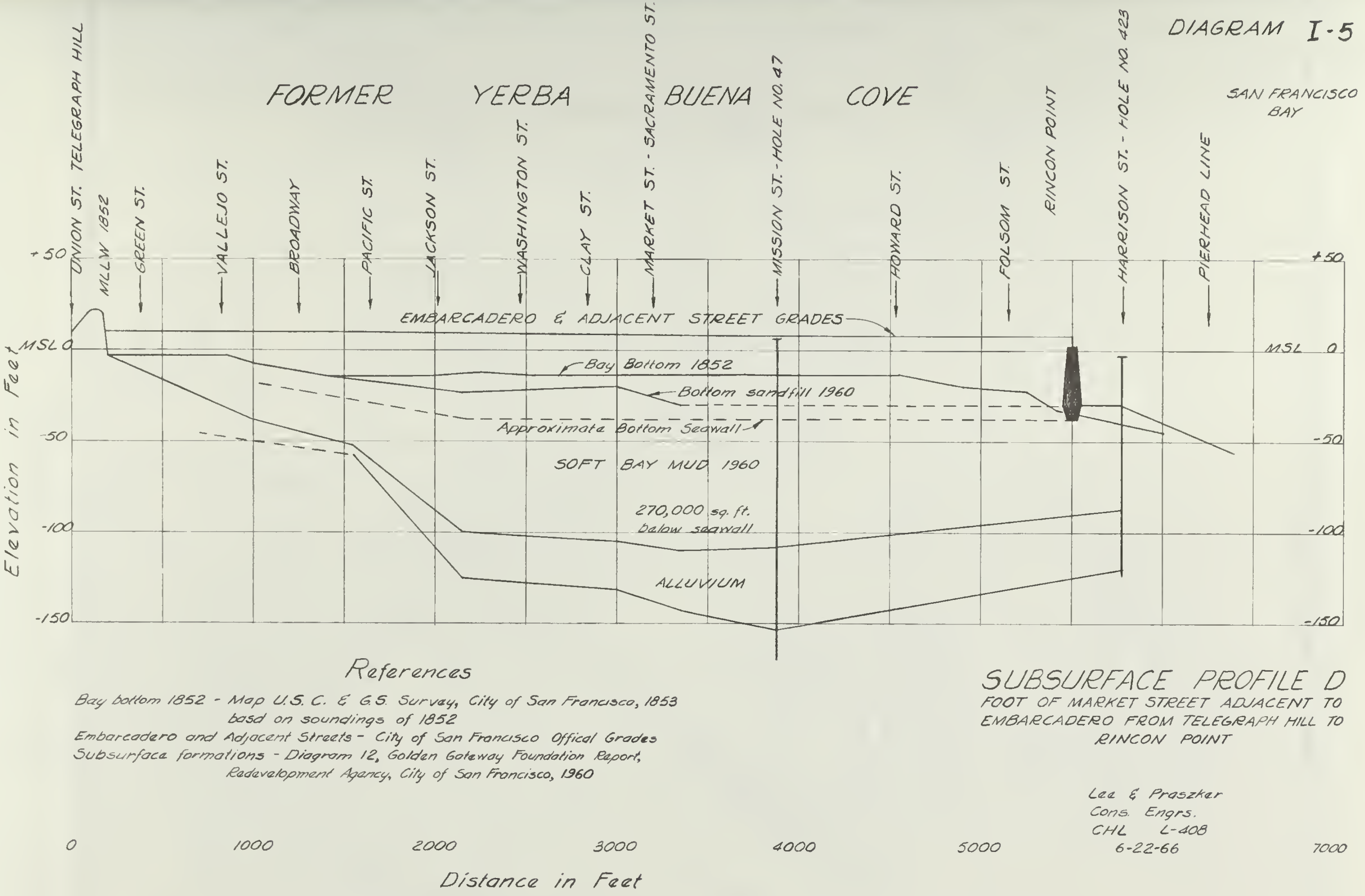


DIAGRAM I-5
FORMER YERBA BUENA COVE
SAN FRANCISCO BAY
UNION ST. TELEGRAPH HILL
MLLW 1852
GREEN ST.
VALLEJO ST.
BROADWAY
PACIFIC ST.
JACKSON ST.
WASHINGTON ST.
CLAY ST.
MARKET ST. - SACRAMENTO ST.
MISSION ST. - HOLE NO. 47
HOWARD ST.
FOLSOM ST.
RINCON POINT
HARRISON ST. - HOLE NO. 423
PIERHEAD LINE
EMBARCADERO & ADJACENT STREET GRADES
Bay Bottom 1852
Bottom sandfill 1960
Approximate Bottom Seawall
SOFT BAY MUD 1960
270,000 sq. ft. below seawall
ALLUVIUM
+50
MSL 0
-50
-100
-150
Elevation in Feet
0
1000
2000
3000
4000
5000
7000
Distance in Feet
References
Bay bottom 1852 - Map U.S.C. & G.S. Survey, City of San Francisco, 1853 basd on soundings of 1852
Embarcadero and Adjacent Streets - City of San Francisco Offical Grades
Subsurface formations - Diagram 12, Golden Gateway Foundation Report, Redevelopment Agency, City of San Francisco, 1960
SUBSURFACE PROFILE D
FOOT OF MARKET STREET ADJACENT TO EMBARCADERO FROM TELEGRAPH HILL TO RINCON POINT
Lee & Praszker
Cons. Engrs.
CHL L-408
6-22-66

Figure 5. Subsidence profile D. Foot of Market Street adjacent to Embarcadero from Telegraph Hill to Rincon Point.

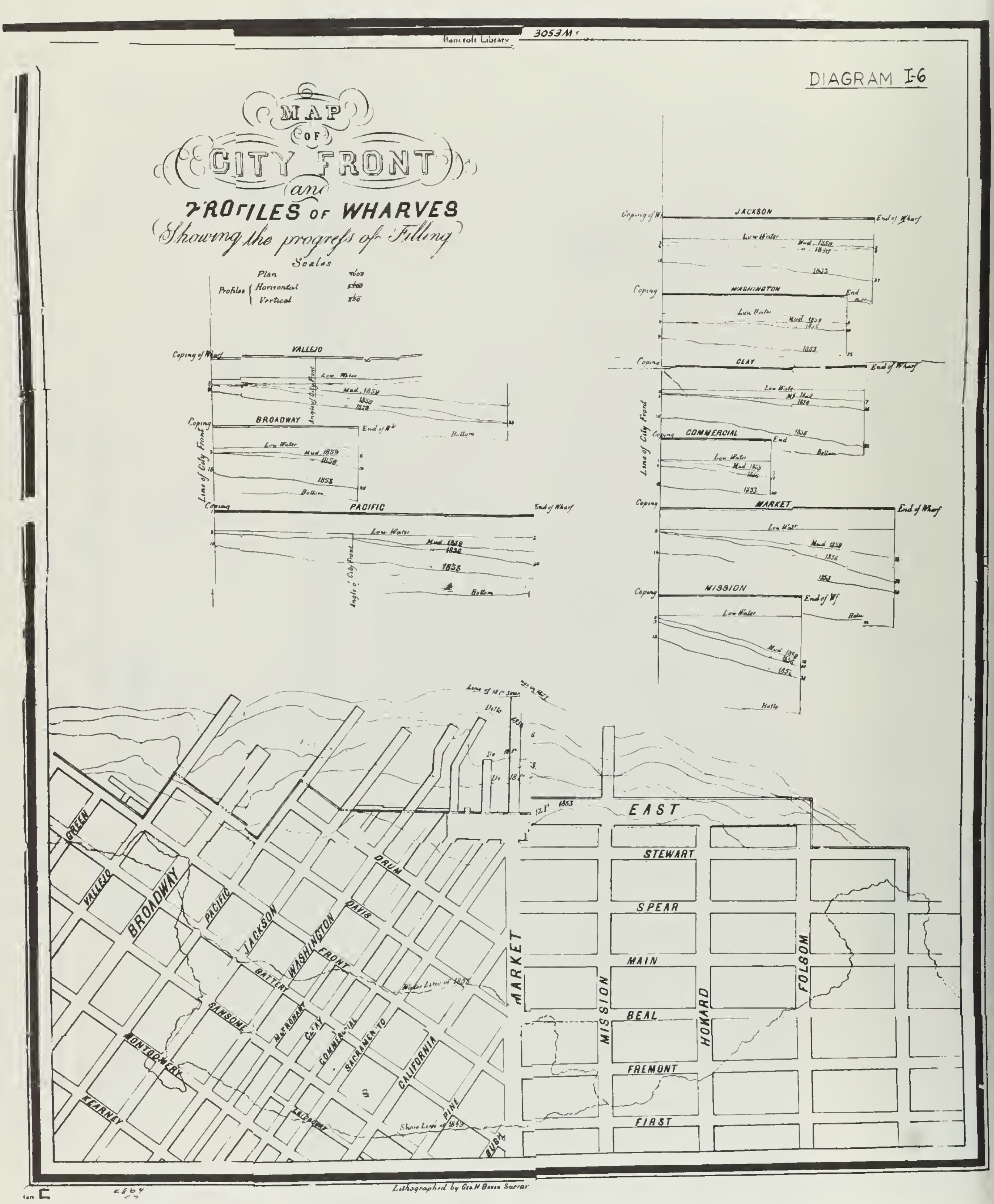


Figure 6. Map of the City Front and profile of wharves showing the progress of filling 1853 to 1859.

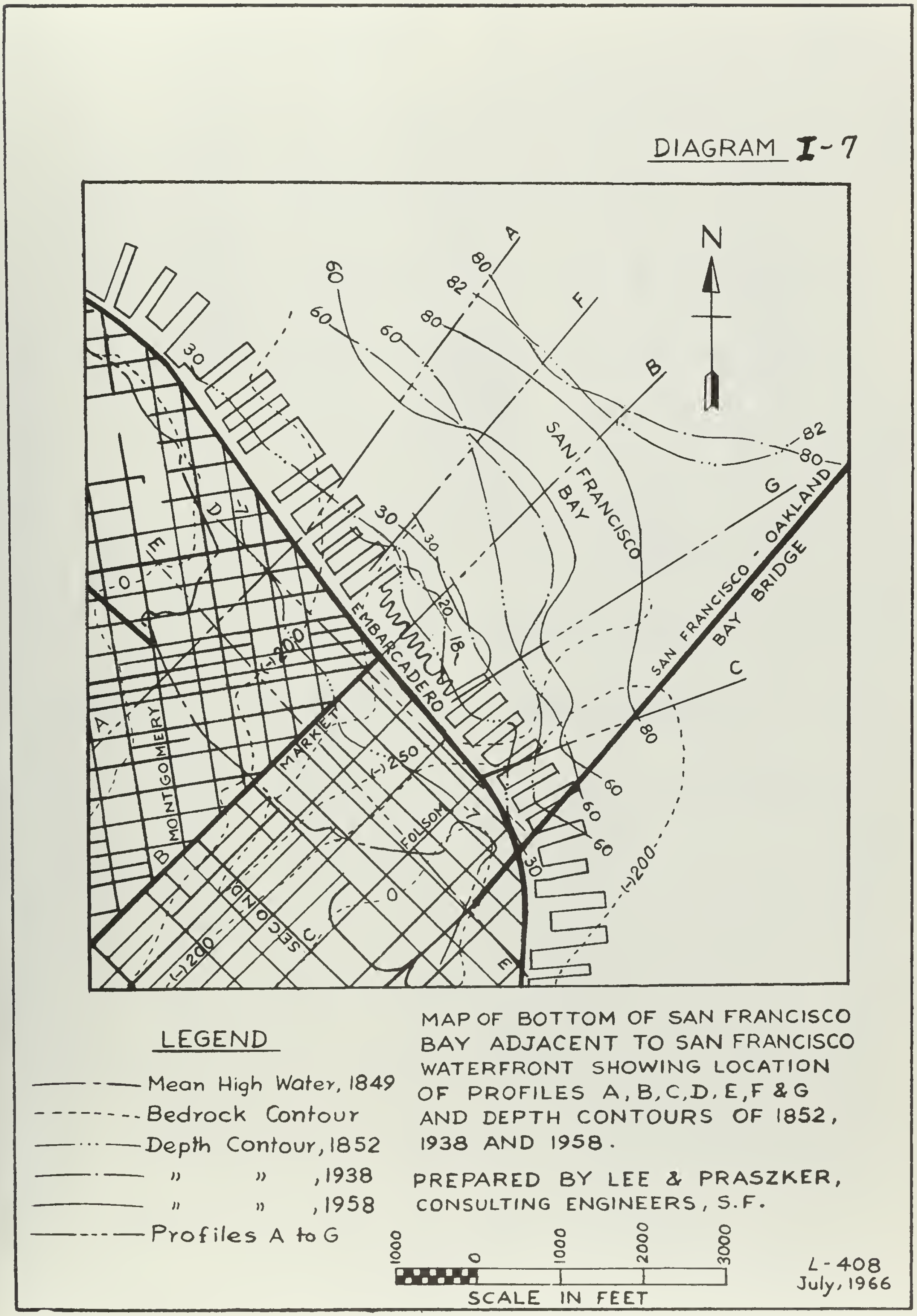


DIAGRAM I-7
N
SAN FRANCISCO BAY
SAN FRANCISCO - OAKLAND BAY BRIDGE
EMBARCADERO
MARKET
MONTGOMERY
FOLSOM
SECOND
A
B
C
D
E
F
G
82
80
60
30
20
18
0
(-)200
(-)250
LEGEND
Mean High Water, 1849
Bedrock Contour
Depth Contour, 1852
" " , 1938
" " , 1958
Profiles A to G
MAP OF BOTTOM OF SAN FRANCISCO BAY ADJACENT TO SAN FRANCISCO WATERFRONT SHOWING LOCATION OF PROFILES A, B, C, D, E, F & G AND DEPTH CONTOURS OF 1852, 1938 AND 1958.
PREPARED BY LEE & PRASZKER, CONSULTING ENGINEERS, S.F.
1000
0
1000
2000
3000
SCALE IN FEET
L-408
July, 1966

Fure 7. Map of bottom of San Francisco Bay adjacent to San Francisco waterfront showing location of profiles A to G and depth contours 1852, 1938, and 1958.

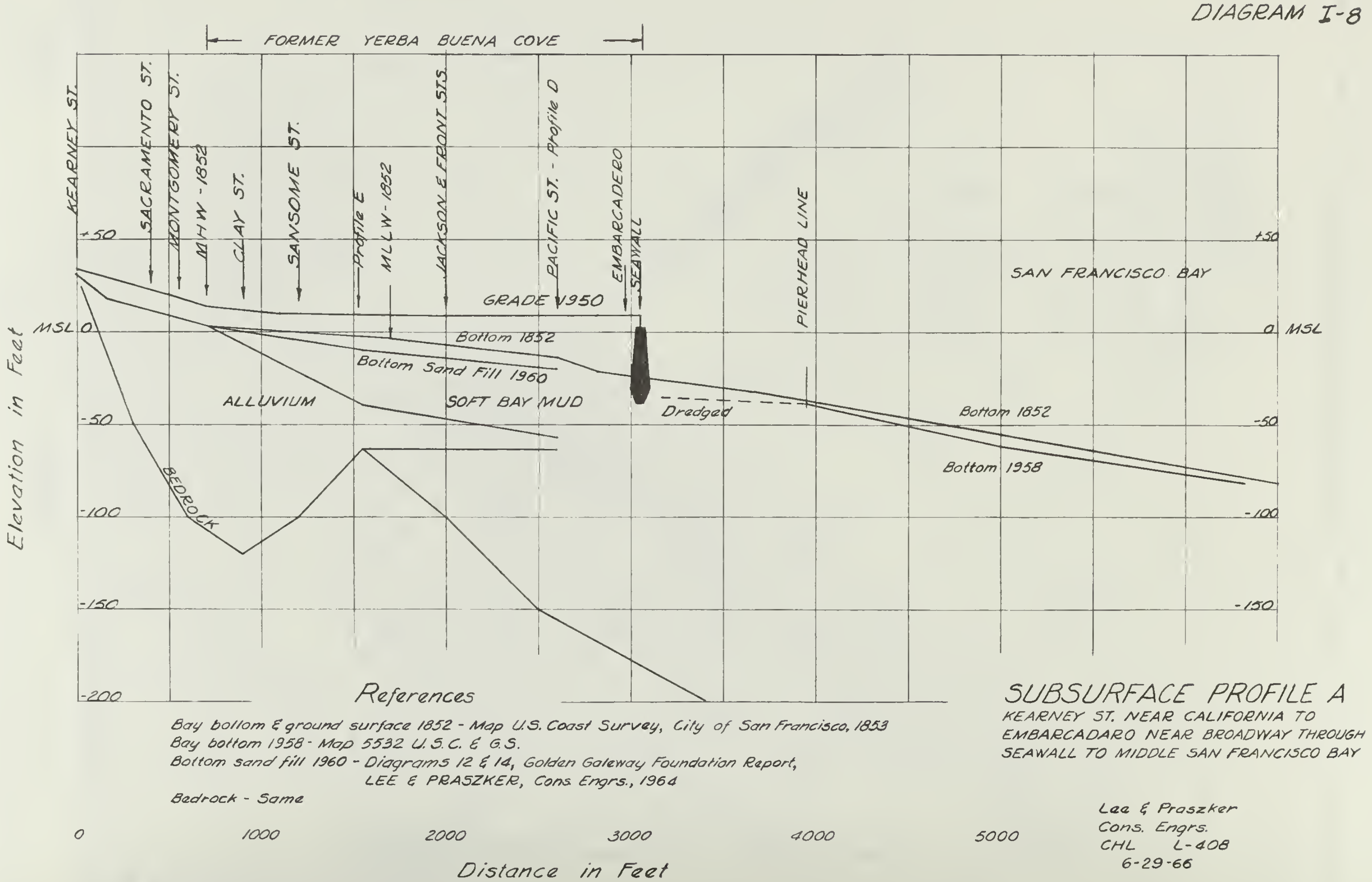


Figure 8. Subsurface profile A. Kearney Street near California to Embarcadero through Broadway.

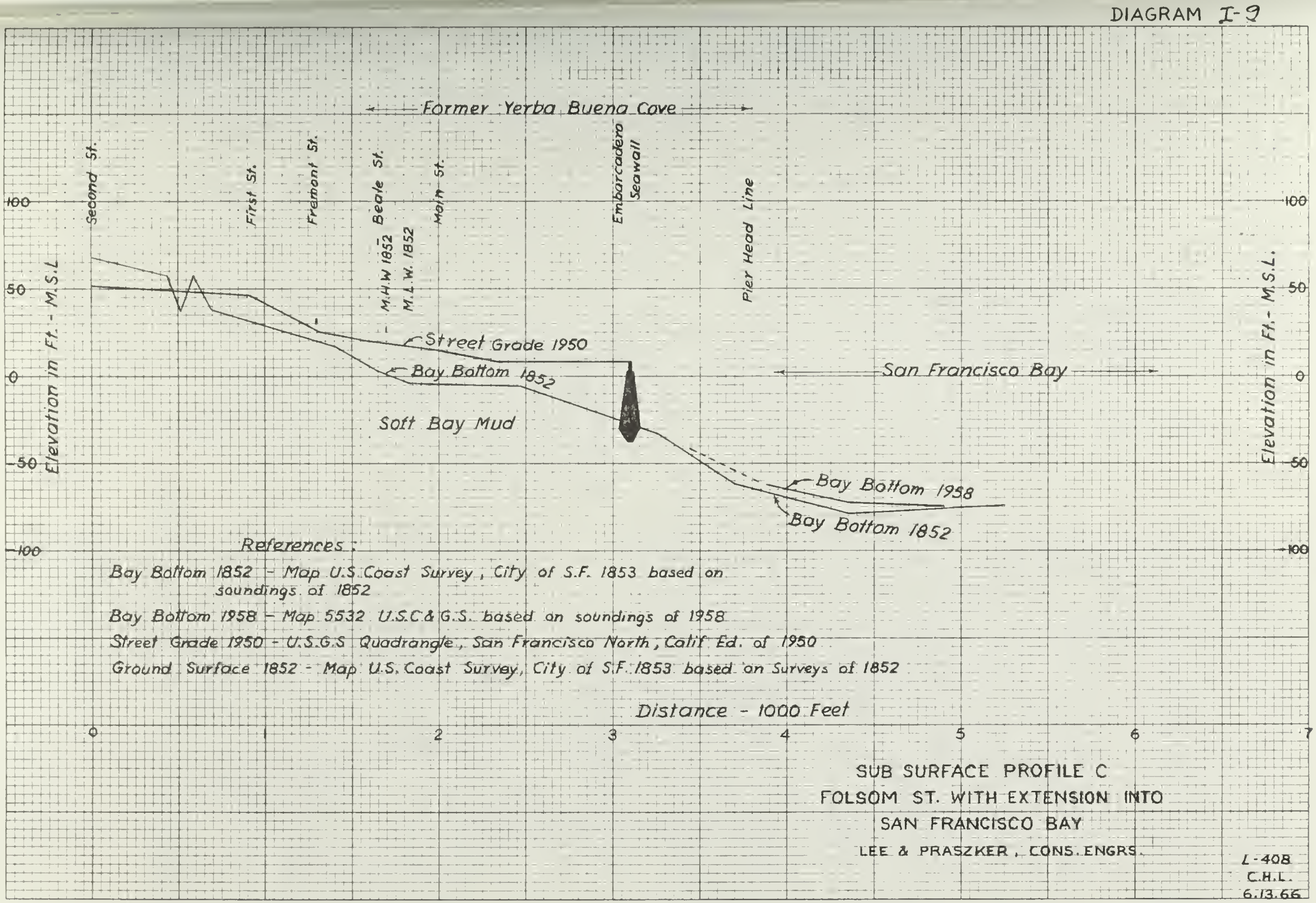


Figure 9. Subsurface profile C. Folsom Street with extension into San Francisco Bay.

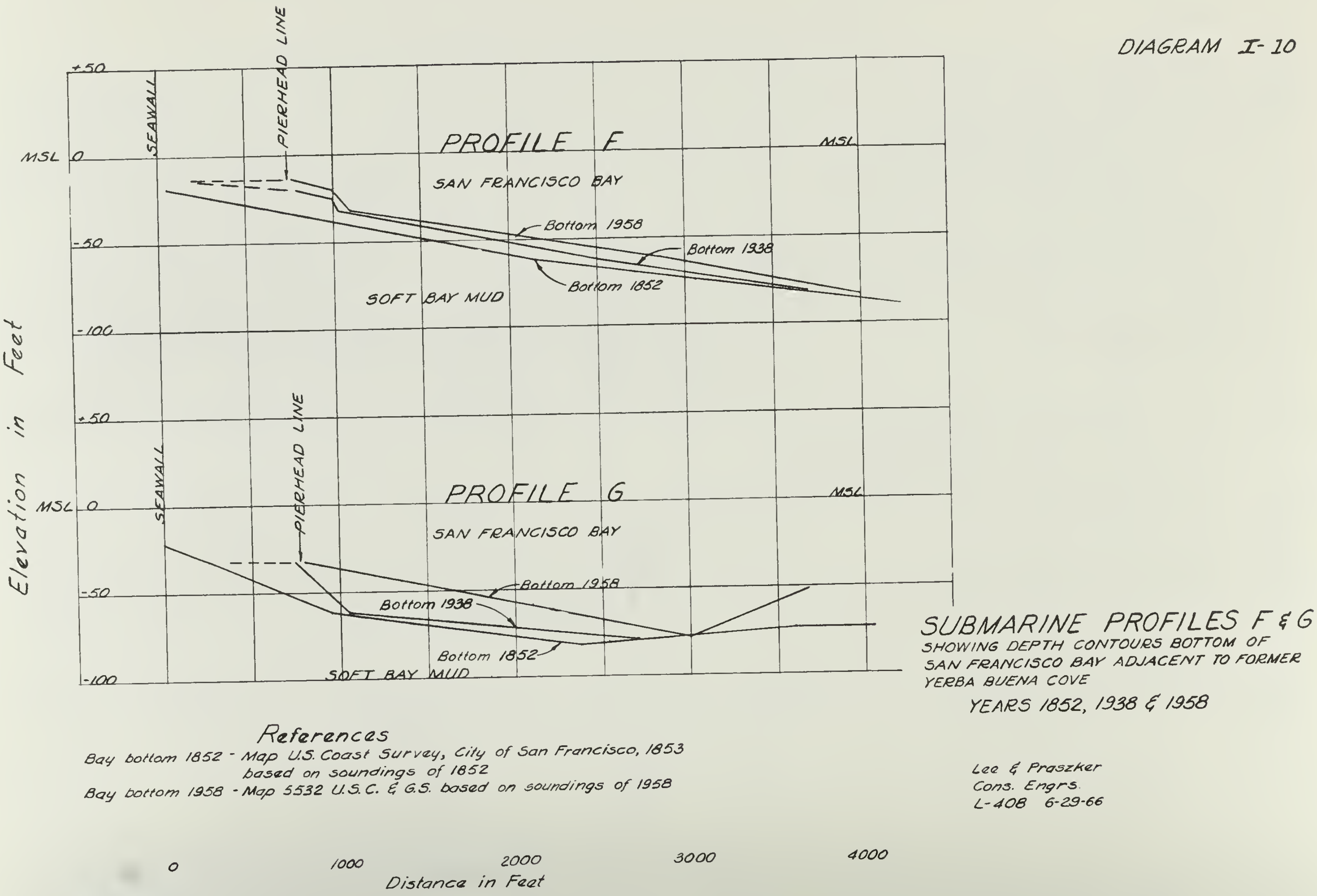


Figure 10. Submarine profiles F and G, showing depth contours bottom of San Francisco Bay adjacent to former Yerba Buena Cove, 1852, 1938, and 1958.

CASE HISTORY 2.

TREASURE ISLAND FILL

By CHARLES H. LEE, Consulting Engineer

ILLUSTRATIONS

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Figure 1.	Preliminary test boring location, Treosure Island, June 1935.
Figure 2.	Typicol rate af settlement curves, Treasure Islond fill, 1937 ta 1964.
Table 1.	U.S.C. & G.S. control manuments Treasure Island.
Table 2.	Compilation of records—rate af settlement af Treasure Island fill.

This fill was placed upon Yerba Buena Shoals, a allow water area exceeding 735 acres, lying north Yerba Buena Island in San Francisco Bay. A small rtion of the area, extending from the northwest int of Yerba Buena Island, was a sand spit, awash low tide near the island, but deepening to 18 feet low Mean Lower Low Water at a distance of 7,000 t. Skirting the sand spit on the east and north, at a pth of 18 to 25 feet below MLLW, was a soft bay ıd deposit overlying the deeply buried former chan- l and ravine of Temescal Creek. This was a major eam draining the east bay hills which, at an earlier ologic period, flowed northwesterly to join the cramento-San Joaquin River north of Angel Island. ne full depth of this mud was not determined by al borings, but extrapolations from adjacent borings ng the line of the San Francisco-Oakland Bay dge indicate the elevation of the ravine bottom to 95 feet below MLLW.[1]

Treasure Island occupies the sand spit and extends t into the mud area between the southeast and thwest corners of the fill. The above-water area the fill is approximately 400 acres, 3410 feet in lth and 5520 feet in length, of which 65 percent s underlain by fine to coarse loose sand, occasionally rbedded with soft sandy mud, mud with sand, dy clay or clay; and 35 percent underlain by soft d (figure 1). Grade elevation of the fill was +13 t above MLLW and that of the sea wall was +14 .

n the design stage there were two concepts for function and composition of the sea wall. The , was to place a sand mound perimeter up to ele- on zero, on top of which the stone levee would constructed to final grade elevation at +14 feet, is to act as a retaining wall for the fill. Where un- lying material was soft along the east and north s and at the southeast and northwest corners, a pre- nary trench was to be excavated and backfilled h sand. Sea wall construction was to commence at south end of the fill and extend north in the form a U, leaving the north end open for escape of ended fines from the dredger fill and also soft mud laced as plastic flow by the weight of the advanc- sand fill.

he other concept visualized the rock wall as a ining dike for the sand fill during construction, to laced in stages as the filling progressed, and after struction to act as backing for a uniform riprapped e from elevation −6 to elevation +14 feet, the latter to afford protection for the slopes against wave action. This plan omitted placing of a sand blanket upon the mud beneath the sea wall and provided for complete enclosure of the proposed fill area with small spillway only at the north end. Class "B" rock was to be used in the sea wall consisting of quarry run rock, maximum size 1,000 pounds, 50 percent weighing 200 pounds or more, not more than 20 percent 3 pounds or less. By reason of financial arrangements this second plan prevailed.

Two experiences during construction are noteworthy.[2] Shortly after completion of the east sea wall a section 500 feet long, near the northerly end, suddenly settled 10 to 14 feet and disappeared from sight. As a result, construction plans were modified by flattening the side slopes of the wall and depositing a mattress of heavy sand (138 pounds per cubic foot) on the bottom beyond the toe of the slope to act as counter-weight and prevent the underlying plastic mud from heaving and thus causing further settlement in the fill. The north sea wall was also modified by trenching for a width of 400 feet to a depth of 20 to 30 feet into the mud bottom and immediately backfilling with heavy sand before placing the first course of the sea wall.

As the marginal dikes neared completion, extremely fine semiliquid mud (85 percent passing a 325 mesh screen) began to accumulate in a kidney-shaped area ahead of the sand fill, amounting at one time to 1,500,000 cubic yards. Toward the end of the filling operation, with less than 400,000 cubic yards to be placed, this mud "stiffened" and would neither move ahead through the spillway nor permit itself to be covered

Table 1. U.S.C. & G.S. control monuments Treasure Island. Location, elevations, and rate of settlement.

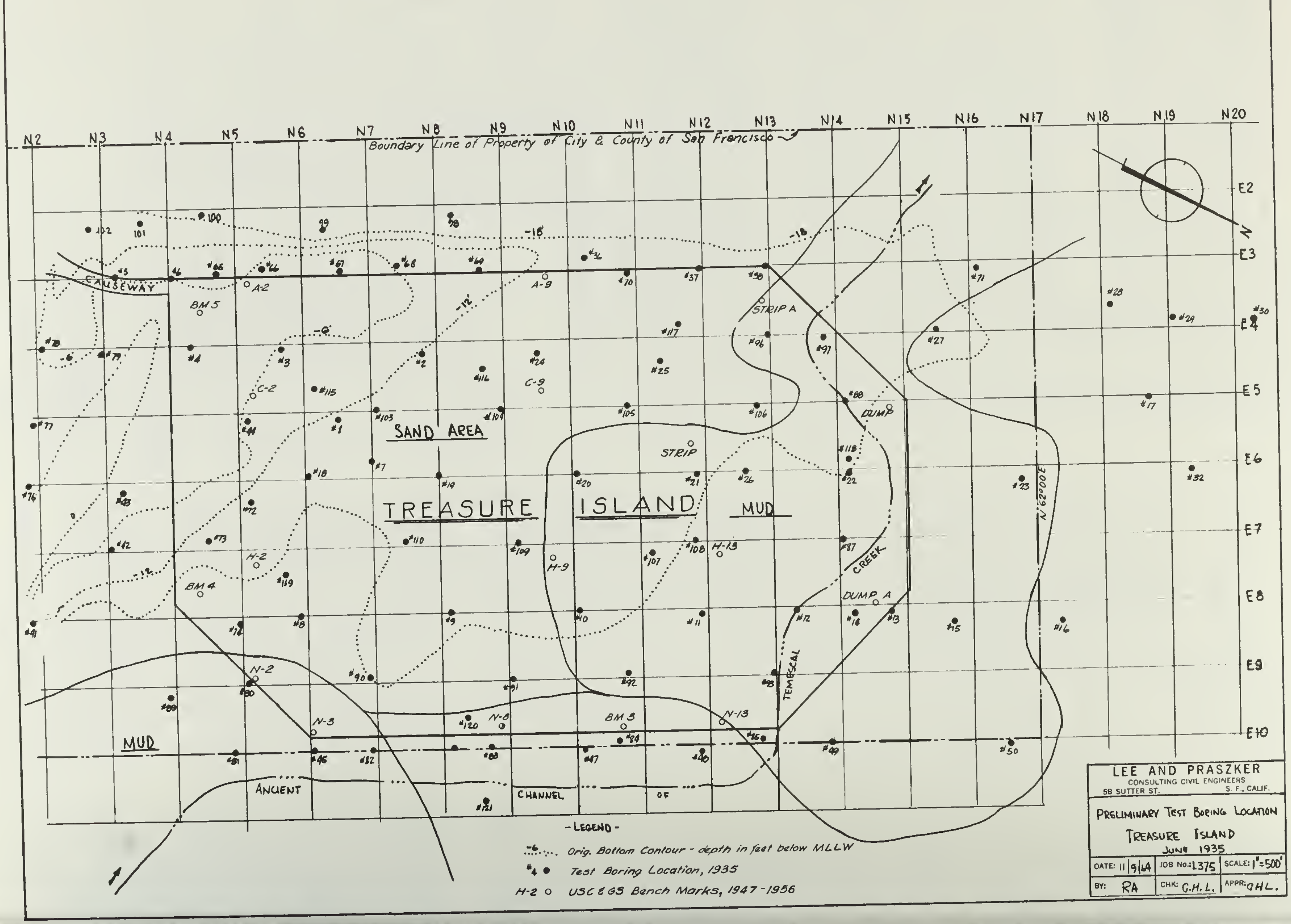
Monument no.	Location	Elevation MSL—feet			Rate of settlement (feet per yr.)
		1947	1956	Difference	
No. 5–1940[1]	At Adm. Bldg. 150′ NE main gate[2]	10.768	10.768	0	0
T1–A2	At T jct. Ave. A & 2nd St.[3]	9.567	9.501	0.066	0.007
T1–C2	At T jct. NE lane Ave. C & 2nd St.[3]	8.717	8.301	0.416	0.046
T1–HZ	At T jct. cl.2nd St. & Ave. H.[3]	7.785	7.165	0.620	0.069
No. 4–1940[1]	At Fire Sta. No. 1, 160′ W. of S.E. Cor. T.1.[4]	9.259	8.934	0.325	0.036
T1–NZ	At T jct. Ave. N & 2nd St. near Pier 14[5]	7.083	6.368	0.715	0.079
T1–N3	At jct. Ave. N. & 3rd St.[3]	8.642	8.599	0.043	0.048
Strip	166′ W. of E. end abandoned landing strip[3]	8.757	7.848	0.909	0.101
Strip A	At W. end—abandoned landing strip[3]	8.832	7.831	1.001	0.111
T1–A9	At T jct. 9th St. & Ave. A.[3]	6.598	5.722	0.876	0.097
T1–C9	At small circular park C. Jct. Ave. C & 9th St.[3]	8.392	7.864	0.528	0.059
T1–H9	At Inters. cl. Ave. H & 9th St.[3]	7.818	7.710	0.108	0.012
T1–N8	At jct. cl. Ave. N & 8th St.[3]	9.203	9.186	0.017	0.002
No. 3–1940[1]	At old Ferry Slip 175′ N. jct. Ave. N. & 10th St.[4]	9.583	9.547	0.036	0.004
T1–N13	138′ NE jct. 13th St. & Ave. N.[3]	7.848	7.808	0.040	0.004
Dump A	91′ S of SE Cor. Pier 22[3]	6.411	6.148	0.263	0.029
Dump		6.102			
T1–H13	At C.T. jct. Ave. H. & 13th St.[3]	6.516	6.102	0.414	0.046

[1] Tidal Mark.
[2] 3.5+ feet higher than driveway.
[3] 1.+ feet lower than street.
[4] At level of street.
[5] 0.95 feet lower than street, paving or black top.

Francisco Bay Exposition Co. Report on Sub-Aqueous Material Underying and Adjacent to the Proposed Dredger Fill Northwest of Yerba Buena Island by Charles H. Lee, Consulting Engineer, 1935.

[2] Report on Yerba Buena School Reclamation Project, San Francisco Bay, California, by Corps of Engineers, U.S. Army, San Francisco District, 1939.

DIAGRAM II-1


N2
N3
N4
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N16
N17
N18
N19
N20
E2
E3
E4
E5
E6
E7
E8
E9
E10
N
Boundary Line of Property of City & County of San Francisco
CAUSEWAY
SAND AREA
TREASURE ISLAND MUD
MUD
ANCIENT CHANNEL OF
TEMESCAL
CREEK
DUMP
DUMP A
STRIP
STRIP A
N 62°00'E
-18
-12
-6
A-2
A-9
C-2
C-9
H-2
H-9
H-13
N-2
N-3
N-8
N-13
BM 3
BM 4
BM 5
102
101
100
99
98
#5
#6
#65
#66
#67
#68
#69
#36
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#37
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#71
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#29
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#78
#79
#4
#3
#2
#116
#24
#117
#25
#96
#97
#27
#17
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#115
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#103
#104
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#11
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#14
#13
#15
#16
#89
#80
#90
#91
#92
#93
#120
#84
#95
#50
#81
#46
#82
#83
#47
#40
#49
#121
-LEGEND-
-6 ..... Orig. Bottom Contour - depth in feet below MLLW
#4 ● Test Boring Location, 1935
H-2 ○ USC & GS Bench Marks, 1947-1956
LEE AND PRASZKER
CONSULTING CIVIL ENGINEERS
58 SUTTER ST.
S. F., CALIF.
PRELIMINARY TEST BORING LOCATION
TREASURE ISLAND
JUNE 1935
DATE: 11|9|64
JOB NO.: 1375
SCALE: 1"=500'
BY: RA
CHK: G.H.L.
APPR: GHL.

by sand. Instead, it kept heaving until the elevation of the top was 5 feet above project grade. All filling operations were then suspended and a small portable dredge was installed within the sea wall to remove the trapped mud. The small dredge advanced southward from the north edge of the mud area, digging to a depth of 6 and 12 feet into the mud. Filling operations were then resumed and as mud built up in front of he fill, it was removed by the small dredge as rapidly as it accumulated. A total of approximately 130,000 cubic yards of mud was thus removed and it was reported that no soft spots remained.

The nature of the fill was determined soon after completion from samples and logs of more than 100 uger holes drilled to 10 feet maximum depth.[3] The ill was found to be composed of layered dredger-sand with frequent admixture of broken shell and occasional wind rows of clay balls in a sand matrix. The predomiant texture was fine sand, occasionally clayey, with ocal areas of medium to coarse sand and gravelly sand. There were occasional thin beds of clay of irregular xtent representing fines which settled out in isolated ools of muddy water during dredger operations.

Of the fine sand, 95 percent or more was held on he 150 mesh sieve. Of the blue clay balls (marine), nly 2 percent was retained on the 200 mesh sieve, while 70 percent of the clayey sand balls (alluvial) was retained on the 200 mesh sieve. The clay balls epresented chunks of cohesive (marine) or semi-coesive (fresh-water alluvial) derived from clay beds ithin the sand, as excavated by dredging, which were ounded in passing through the delivery pipe line. he dry weight of sand fill ran 95 to 100 pounds per ubic foot, void ratio 0.75 to 0.80 and permeability 3 to 283 gallons per square foot per 24 hours at 100 ercent gradient and 60° F. temperature. Clay balls n 60 to 65 pounds per cubic foot dry weight and 64 void ratio. They were impermeable.

Total volume of material handled by dredges was 9,382,000 cubic yards of which 20,947,000 cubic ards, or 71.5 percent, was retained as fill, the remainer being lost either by tidal current erosion, or by otation of fines. The loss of sand by erosion has been onservatively estimated as 100,000 cubic yards. Heavy and deposited from hoppers outside the easterly sea wall was found to have penetrated 9 feet below origil bottom.

Filling commenced February 11, 1936 and was comleted July 2, 1937, except for refill operations August 1 to 24, 1937. Fill material was obtained from various ources, as follows:

	Cubic Yards
"*Heavy sand*", coarse and well-graded sand and gravel from Presidio, Alcatraz and Knox Shoals, transported by hopper dredges	1,261,000
"*Soft blue marine sand*", very fine to fine sand, 70 to 80 percent held by 200 mesh screen, obtained from channel south of fill and transported by pipe line from dredges	6,348,900
"*Soft blue marine sand*", fine textured, 50 to 60 percent held on 200 mesh sieve, lying east and north of fill, transported by pipeline from dredges	14,355,900
"*Yellow alluvial sand*", cohesive, well-graded fine to medium 98 percent held on 150 mesh sieve, from east bay area transported by pipeline dredge. Contained lenses of clayey sand which were discharged as "clay balls". Soft bay mud overburden of 5 to 20 feet in depth east of county line was wasted	7,142,574
"*Miscellaneous*" material, principally black sandy mud, handled by clam shell dredge over the sea wall	251,758
Total	29,360,132

Mass settlement rate of the fill after completion to grade has been uneven. The result of observations made during 1937 and 1938 by the San Francisco Bay Exposition Company are indicated in the first four columns of table 2. Later observations were made by the U.S. Coast and Geodetic Survey which established 18 bench marks on Treasure Island and observed elevations in 1947 and 1956 (table 1). The results for typical bench marks located in designated portions of the fill are shown in column 5 of table 2. Measurements by the District Public Works Office, 12th Naval District, in connection with building construction are shown in column 6 of table 2.

Table 2. Compilation of records—rate of settlement of Treasure Island fill, 1937 to 1964.

Location on fill	1937[1]		1938[1]		1947–56[2]	1957–64[3]
	Ft. per mo.	Ft. per yr.	Ft. per mo.	Ft. per yr.	Ft. per mo.	Ft. per yr.
Southwest Portion	.086	1.05	.050	0.60		
U.S.C. & G.S.B.M. T1–A2	--	--	--	--	0.007	
U.S.C. & G.S.B.M. T1–C2	--	--	--	--	0.046	
Northwest Portion	0.13	1.56	.044	0.53		
U.S.C. & G.S.B.M. Strip A	--	--	--	--	0.111	0.063
U.S.C. & G.S.B.M. Strip	--	--	--	--	0.101	0.015
Northeast Portion	0.13	1.56	.064	0.77		
U.S.C. & G.S.B.M. Dump A	--	--	--	--	0.029	
U.S.C. & G.S.B.M. T1–H13	--	--	--	--	0.046	
Southeast Portion	0.15	1.80	.040	0.48		
U.S.C. & G.S.B.M. T1–H2	--	--	--	--	0.069	
U.S.C. & G.S.B.M. No. 4 Tidal	--	--	--	--	0.036	
U.S.C. & G.S.B.M. T1–N2	--	--	--	--	0.078	
U.S.C. & G.S.B.M. T1–N3	--	--	--	--	0.048	
Central Portion						
U.S.C. & G.S.B.M. T1–C9	--	--	--	--	0.059	
Barracks Mess Hall Ave. 15 points	--	--	--	--	--	0.023
BOQ Ave. 13 points	--	--	--	--	--	0.022
U.S.C. & G.S.B.M. T1–H9	--	--	--	--	0.012	
West Sea Wall						
U.S.C. & G.S.B.M. T1–A9	--	--	--	--	0.097	
East Sea Wall						
U.S.C. & G.S.B.M. T1–N13	--	--	--	--	0.004	
U.S.C. & G.S.B.M. T1–N8	--	--	--	--	0.002	
U.S.C. & G.S.B.M. No. 3 Tidal	--	--	--	--	0.004	

[1] Eng. News-Record, Oct. 13, 1938.

[2] Vertical Control Data, U.S.C. & G.S., Diagram NJ 10–8, San Francisco, published Jan. 1964.

[3] Dept. of Navy, Bureau Yards & Docks, Dist. P.W.O., 12th Naval District, San Bruno, California, DPWO Dwg. No. C-45026, rev. to Nov. 1964.

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port on Soil Drainage and Leaching at Treasure Island by Charles H. Lee, Chief of Division of Water Supply and Sanitation. Golden Gate International Exposition, 1938.

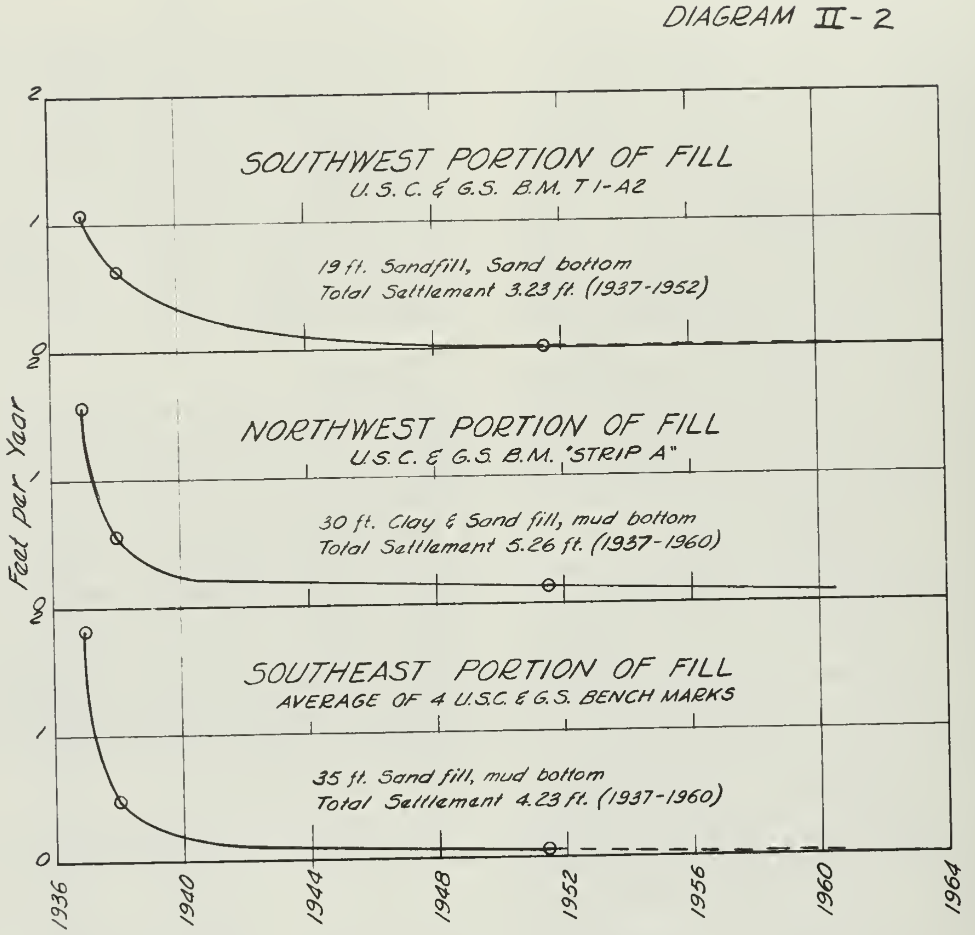

Figure 2. Typical rate of settlement curves, Treasure Island fill, 1937 to 1964.

Typical rate of settlement curves have been plotted from this data representative of the southwest, northwest and southeast portions of the fill (figure 2). In the southwest portion, where 19 feet of sand fill was placed upon sand bottom, settlement has been the least and had practically ceased by 1956 when it amounted in total to 3.23 feet. Most of the settlement took place during the first 7 years.

In the northwest portion, where 30 feet of clay and sand fill was placed upon deep mud bottom, total settlement to 1960 has amounted to 5.26 feet with a current rate of settlement of 0.063 foot per year. Most of the settlement occurred during the first four years.

In the southeast portion, where 35 feet of sand fill was placed upon mud bottom, total settlement has been 4.23 feet with a current rate (projected) of 0.04 foot per year. Most of the settlement occurred during the first five years.

Back of the east sea wall where the sand counterweight was placed outside of the wall, the settlement rate 1947 to 1956 was nominal indicating complete stability. Back of the west sea wall, settlement during the same period was at a rate of twice that for the main fill, indicating the occurrence of sand seepa through the rock levee induced by exposure to hea wave action. Local areas of excessive sand seepa occurred here during the exposition and were bac filled.

CASE HISTORY 3. SETTLEMENT OF FILL IN AND AROUND BUILDINGS ON PILED FOUNDATIONS

Supply Annex, U.S. Naval Air Station, Alameda, Califor

By EDGAR BECKER, Project Engineer, Lee & Praszker

(Figures and photos follow Page 74)

The Supply Annex is located in the northeast cor of the U.S. Naval Air Station, Alameda, Califor between Main and Webster Streets just east of M Street and approximately 1250 feet north of Atla Avenue as shown on figure 1. The Supply Anne generally rectangular in shape with maximum dim sions of approximately 1250 feet by 3100 feet. seven major structures located within its bounda re designated by the Navy as Buildings Nos. 361, 364, 65 and 368 through 371 and are all used for storage f supplies.

Prior to approximately 1925 the area now occupied y the Supply Annex was part of a tidal marsh with oughs as shown in figure 1. Between the years 1925 nd 1929 the area (as well as adjacent areas) was filled y intermittent dredger fill generally consisting of feet to 14 feet in thickness of gray, silty clay with inor admixtures of sand, and a sand topping 4 feet 8 feet thick as shown on figures 2 and 3. All this ll was placed on the marshlands and sloughs which ere underlain by soft mud, characteristic of San rancisco Bay, to varying depths as shown on figures and 3. Figure 1 shows the mud thickness contours hich thickness includes mud fill, all as obtained from st boring data. It is to be noted that the thread of mud-filled ravine approximately parallels the north oundary of the Supply Annex and extends eastward d westward thereof.[1] No fill has been placed in this neral area since 1929.

In 1929 The San Francisco Bay Airdrome, Inc., ased the property from the then owner, the Univer- ty of California. Two structures were built by the irdrome in 1930–31, one of which remains as build- g No. 365; the other has been cut in half and moved d is now building No. 364. The airdrome was in eration until 1941. Between the years 1941 and 1953 e Navy acquired the property and in 1953 completed e construction of buildings Nos. 368–371. This his- ry relates in particular to these four buildings and ilding No. 365.

Building 365.

Building 365 was constructed by San Francisco Air- ome, Inc., in 1930-31. Only the structure was sup- rted on piles, the floor having been constructed on ade. The structure is in relatively good condition d inasmuch as the building is on piles it can be sumed that the structure itself has not undergone preciable settlement, although photo 1 shows slight evenness of hangar door trackway due to differen- al settlement of approximately 4 inches. However, e grounds around the building have settled approxi- ately 2.25 feet below the elevation of 1930–31 as own in photo 2. Photos 1 and 2 also show remedial ork which has been done to provide entrance to the ilding. In 1962 the Navy completed a reconstruc- n of the interior floor which had settled to a maxi- m of 3 feet due to imposed storage loads. This con- sted of fill, base rock and asphalt paving, to bring e building floor to the level of 1930–31, as indicated column bases and door runway. Following recon- uction, settlement readings were taken for three ars from which settlement contours have been awn as shown on figure 4, indicating a maximum sidence of 0.8 foot. It is to be noted that this settlement is not due alone to the weight of the refilling but also to the weight of the material stored in the warehouse. The load which caused this settlement cannot be estimated. Assuming a similar loading condition to obtain in the future as that which caused the above three-year settlement, a settlement-time curve has been prepared which shows an estimated maximum subsidence of 1.4 feet by the year 1971. Photo 3 shows the new floor breaking away from the portion of the floor constructed over the pile cap which supports the building column.

Buildings 368–371

In 1953 buildings 368–371 were completed. These also are supported on piles except for their floors which are supported on grade. Settlement readings were taken on the floors for 9 years and indicated an average subsidence of approximately 1.25 feet and a maximum of 1.68 feet. This subsidence was due to the building floor pad fill varying from 2 to 5 feet in thickness and the weight of the materials stored. At present the average settlement is approximately 1.5 feet as shown in photo 4 which was taken at an outside building corner. An interesting feature of these buildings is that the settlement was anticipated as was the fact that it would cause settlement for some distance beyond the limits of the building. In order to provide proper drainage of surface water after settlement, the exterior concrete slab was sloped away from the building. This can be seen in photo 5 where a portion of the concrete slab settled for some distance and then "hung-up" on the grade beam and has broken away from the remainder of the slab which followed the ground surface further downward. Extensive repairs have been made for this condition as indicated by asphalt paving adjacent to the buildings as shown on photos 4, 6, 7 and 9. (Photo 8 shows the concrete slab intact.)

Photos 6, 7, 8 and 9 show repairs at entrance to the building to correct for the ground settlement that has taken place. In photo 7 note that the pedestrian doorway is no longer in use.

A time settlement analysis for these building floors indicates that they could settle as much as 2½ feet by 1976 if a similar loading pattern continues as existed for the 9-year period during which readings were taken.

An interesting phenomenon took place during the first two years after completion of the buildings in the eastern quarter of the area between buildings 370 and 371, namely that the ground surface actually heaved 3 inches. This was due to an overloading of the mud underlying the building floors causing it to flow plastically towards the non-loaded area between the buildings.

In conclusion it must be pointed out that these buildings have remained functional in spite of the settlement of the surrounding and interior areas because it was anticipated and the design made accordingly.

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[1] e also "Engineering Geology of San Francisco Bay, California", by Parker Trask, Bulletin of the Geological Society of America, Vol. 62, pp. 1079–1109, 1951.

ILLUSTRATIONS

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75	Figure 1.	Outline map northeast portion U.S. Naval Air Station, Alameda, California.
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82	Photo 6.	Building 369—typical pedestrian porch repair.
83	Photo 7.	Building 369—typical vehicular door ramp repair. Note pedestrian doorway no longer in use.
83	Photo 8.	Building 369—typical vehicular door ramp repair. Note concrete slab adjacent to building intact.
84	Photo 9.	Building 369—typical repair with asphalt paving adjacent to building where "hung-up" concrete slab has been removed. Also note vehicular door ramp repairs.

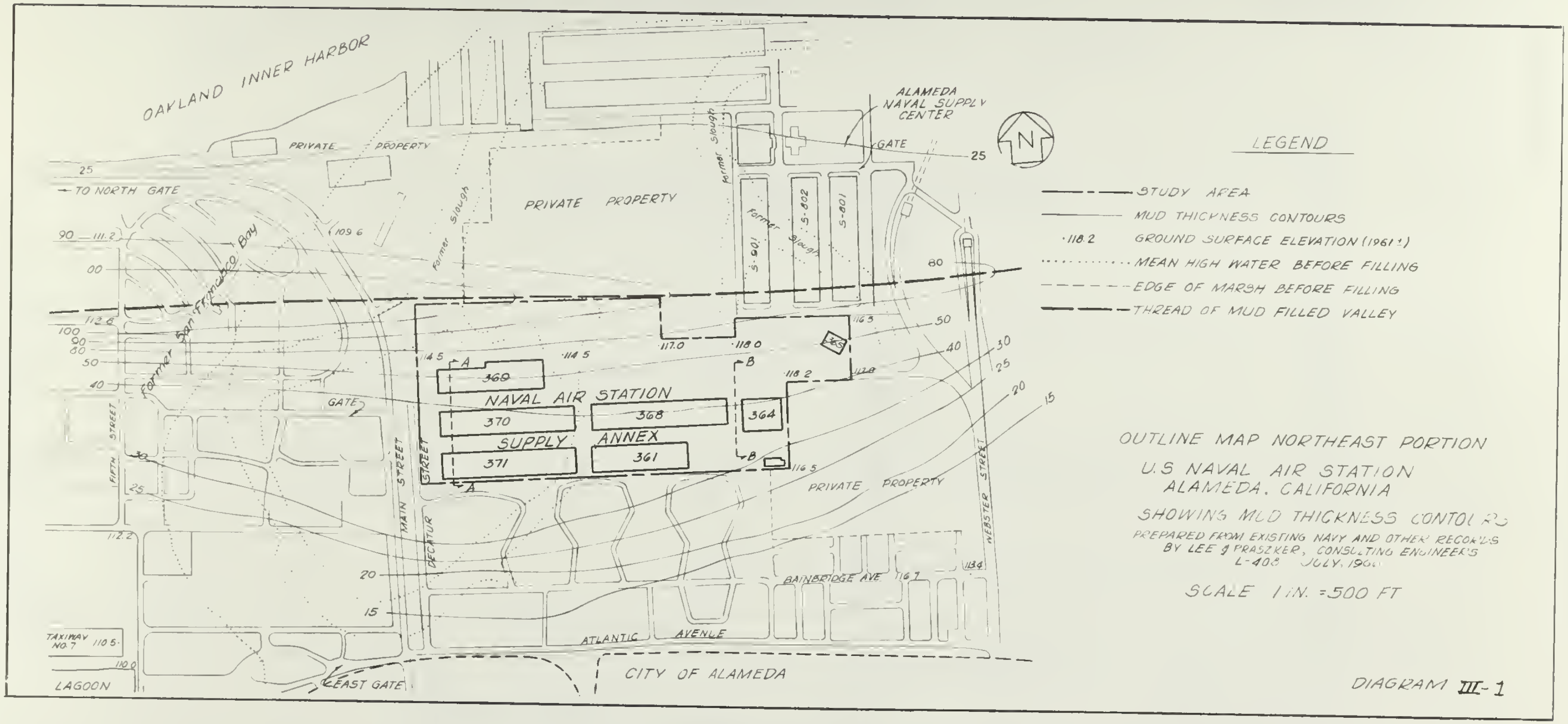


Figure 1. Outline map northeast portion U.S. Naval Air Station, Alameda, California, showing mud thickness contours.

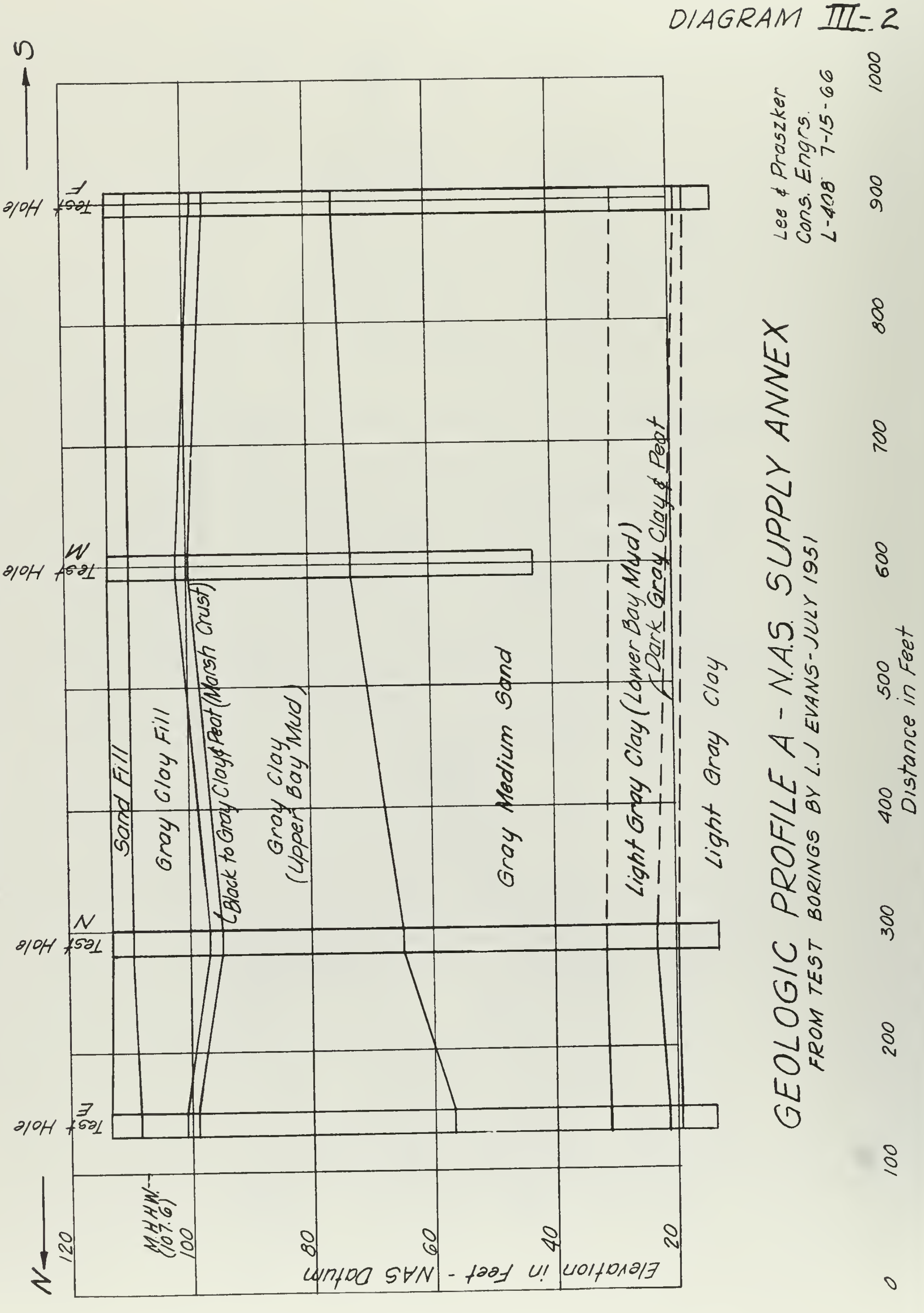


Figure 2. Geologic profile A—Naval Air Station Supply Annex.

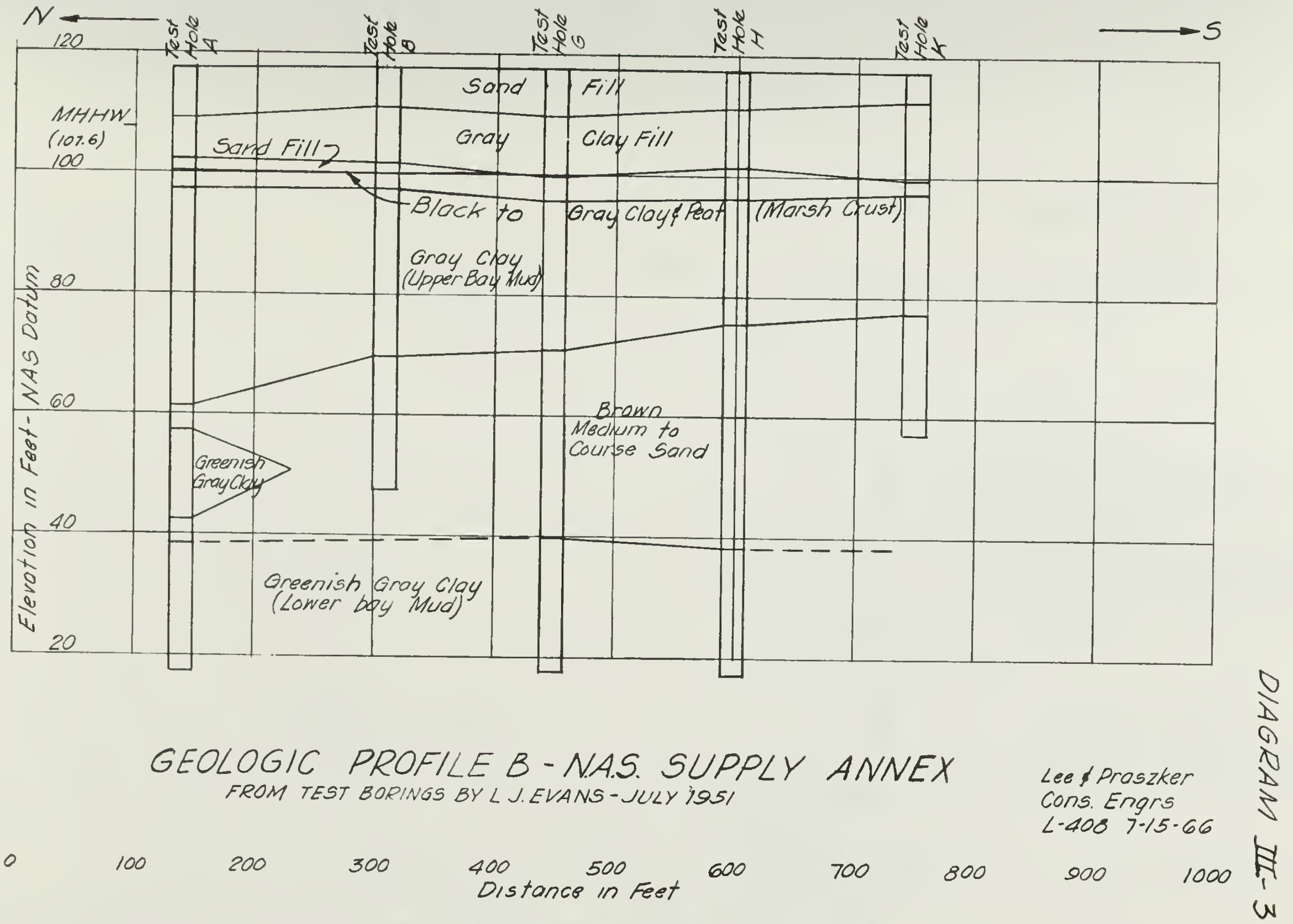
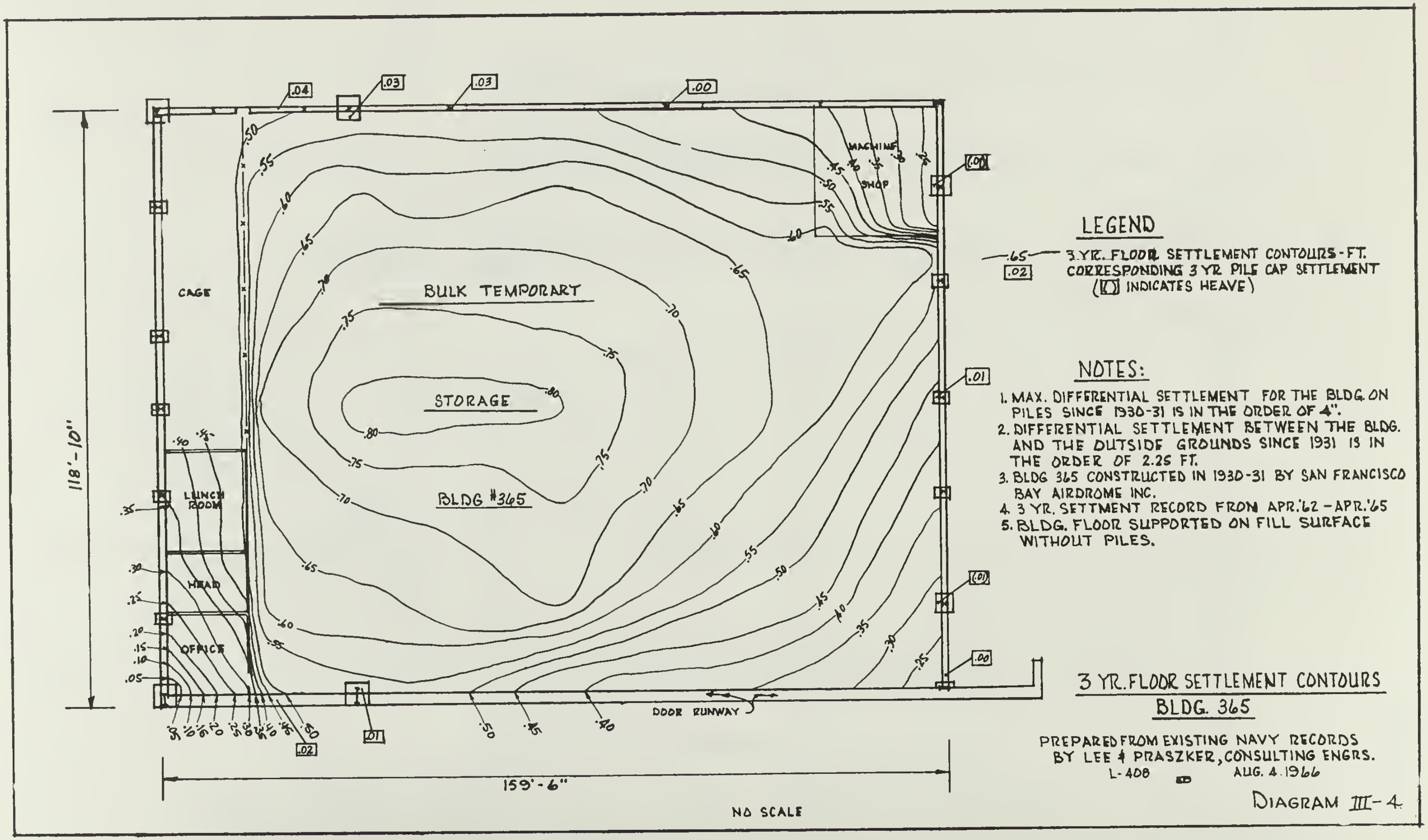

N
S
Test Hole A
Test Hole B
Test Hole G
Test Hole H
Test Hole K
120
MHHW (107.6)
100
80
60
40
20
Elevation in Feet - NAS Datum
Sand Fill
Gray Clay Fill
Sand Fill
Black to Gray Clay & Peat (Marsh Crust)
Gray Clay (Upper Bay Mud)
Brown Medium to Course Sand
Greenish Gray Clay
Greenish Gray Clay (Lower bay Mud)
GEOLOGIC PROFILE B - N.A.S. SUPPLY ANNEX
FROM TEST BORINGS BY L J. EVANS - JULY 1951
Lee & Praszker
Cons. Engrs
L-408 7-15-66
DIAGRAM III-3
0
100
200
300
400
500
600
700
800
900
1000
Distance in Feet

Figure 3. Geologic profile B—Naval Air Station Supply Annex.



LEGEND

.65 3 YR. FLOOR SETTLEMENT CONTOURS - FT.
.02 CORRESPONDING 3 YR. PILE CAP SETTLEMENT
(( ) INDICATES HEAVE)

NOTES:

1. MAX. DIFFERENTIAL SETTLEMENT FOR THE BLDG. ON PILES SINCE 1930-31 IS IN THE ORDER OF 4".
2. DIFFERENTIAL SETTLEMENT BETWEEN THE BLDG. AND THE OUTSIDE GROUNDS SINCE 1931 IS IN THE ORDER OF 2.25 FT.
3. BLDG 365 CONSTRUCTED IN 1930-31 BY SAN FRANCISCO BAY AIRDROME INC.
4. 3 YR. SETTMENT RECORD FROM APR.'62 - APR.'65
5. BLDG. FLOOR SUPPORTED ON FILL SURFACE WITHOUT PILES.

3 YR. FLOOR SETTLEMENT CONTOURS
BLDG. 365

PREPARED FROM EXISTING NAVY RECORDS
BY LEE & PRASZKER, CONSULTING ENGRS.
L-408 AUG. 4. 1966

DIAGRAM III-4

Figure 4. Three-year floor settlement contours, Building 365.

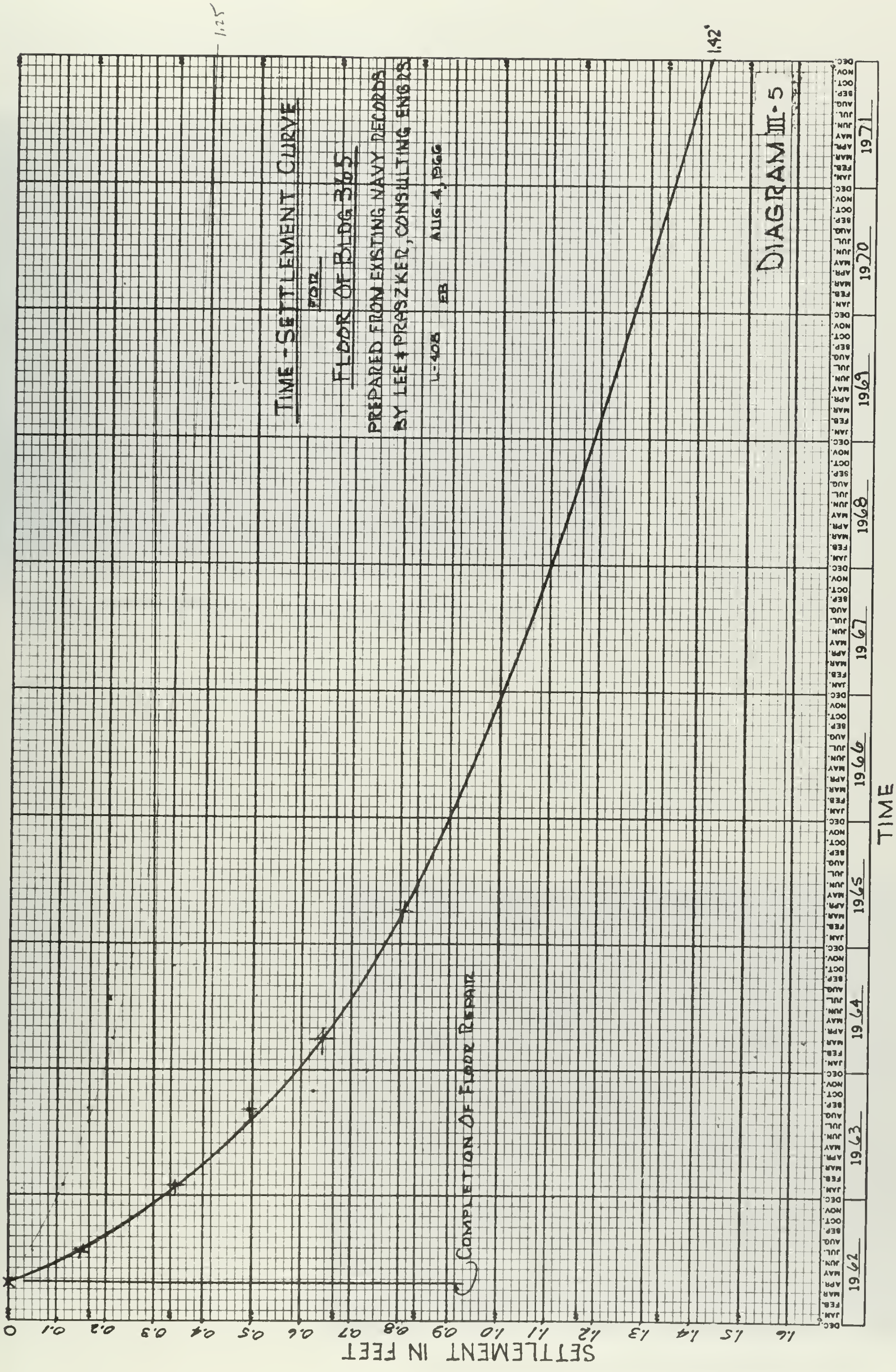


Figure 5. Time-settlement curve for floor of Building 365.

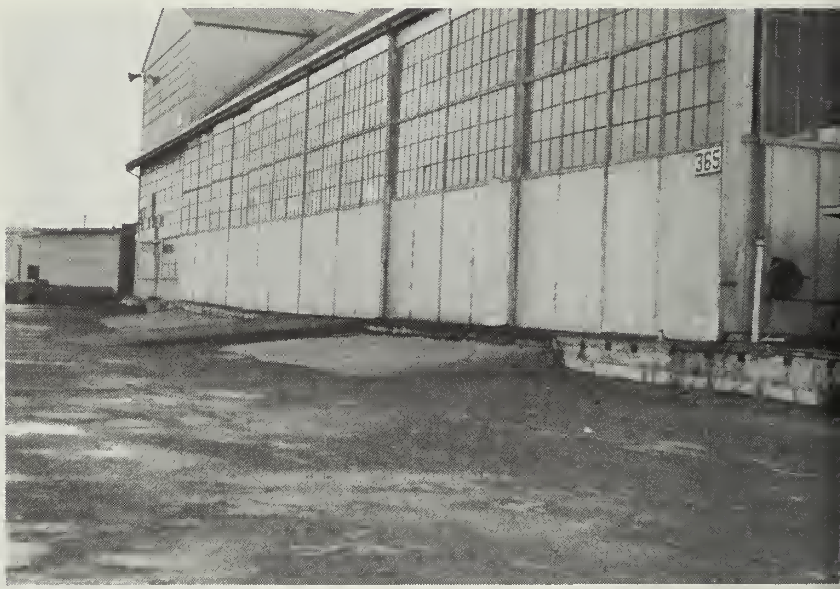


Photo 1. Building 365—showing differential settlement of door trackway.

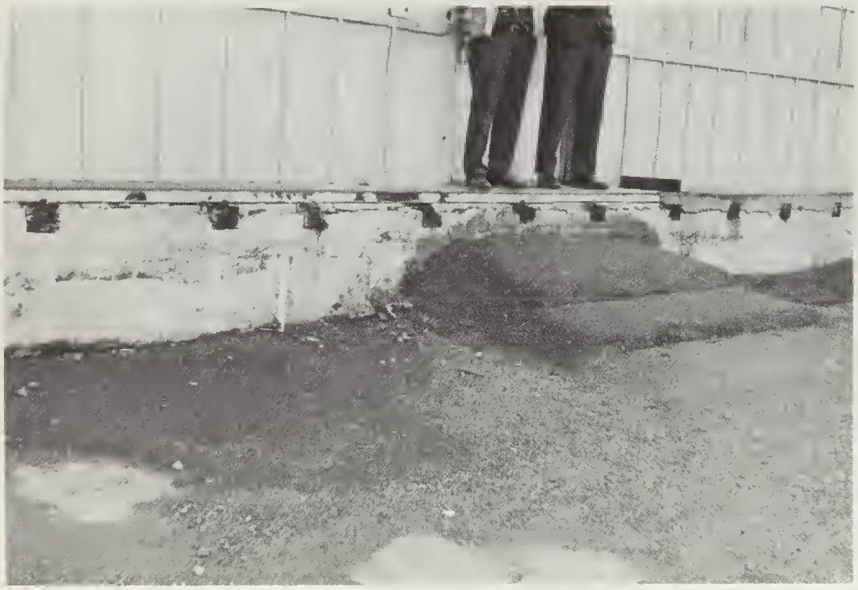


Photo 2. Settlement adjacent to Building 365. Note one-foot rule.

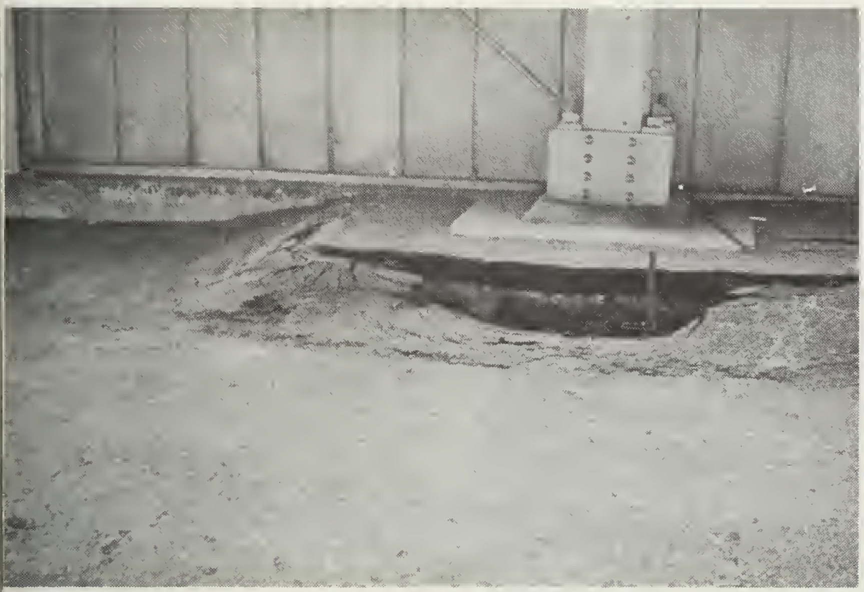


Photo 3. Building 365—floor breaking away from the portion of the floor constructed over pile cap.

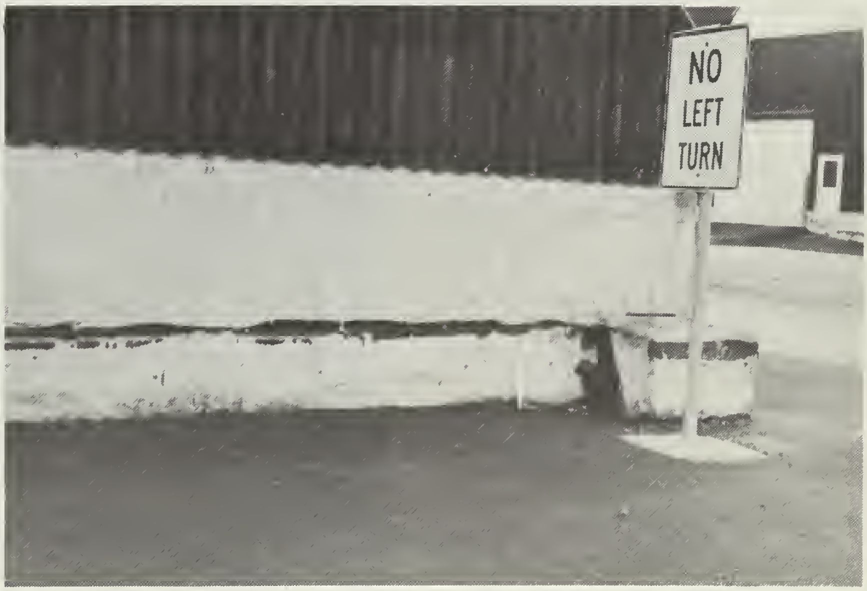


Photo 4. Building 369—1.5-foot settlement of adjacent ground surface.

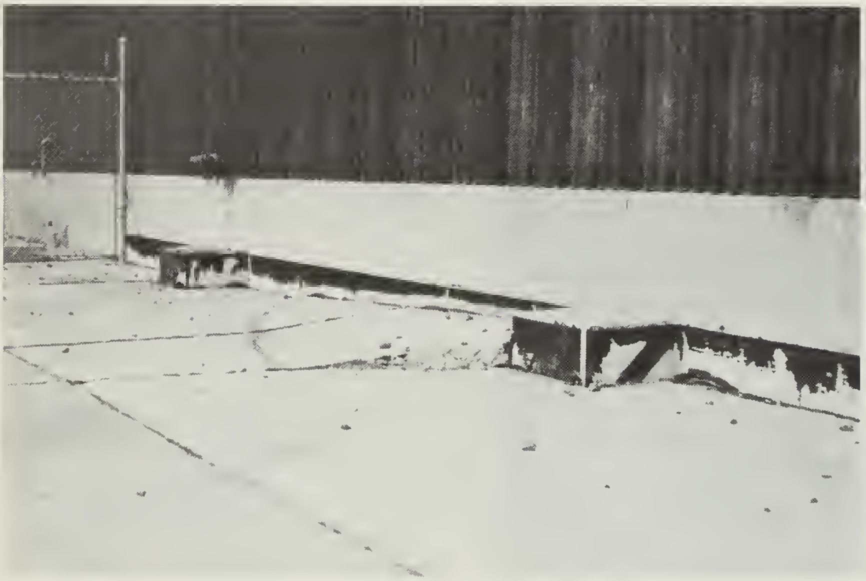


Photo 5. Building 369—showing a portion of exterior concrete slab which "hung-up" on the grade beam.

Photo 6. Building 369—typical pedestrian porch repair.

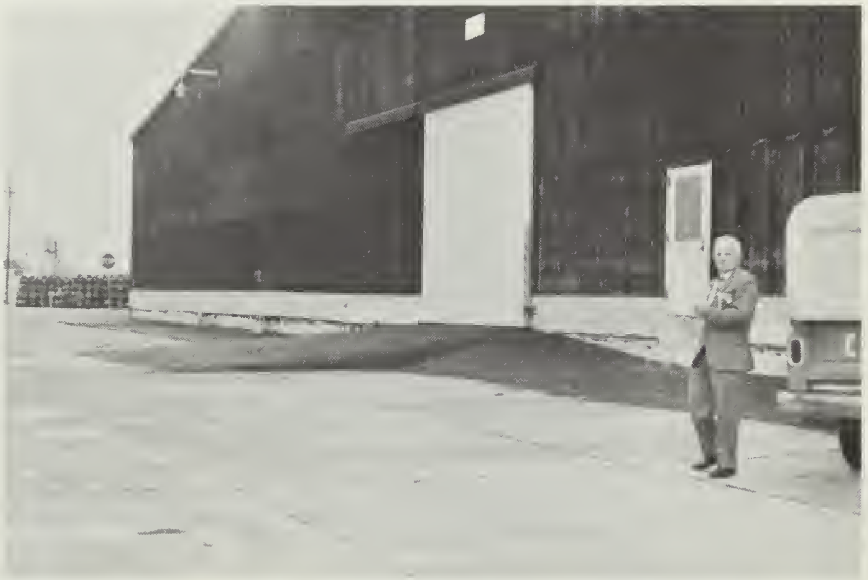


Photo 7. Building 369—typical vehicular door ramp repair. Note pedestrian doorway no longer in use.

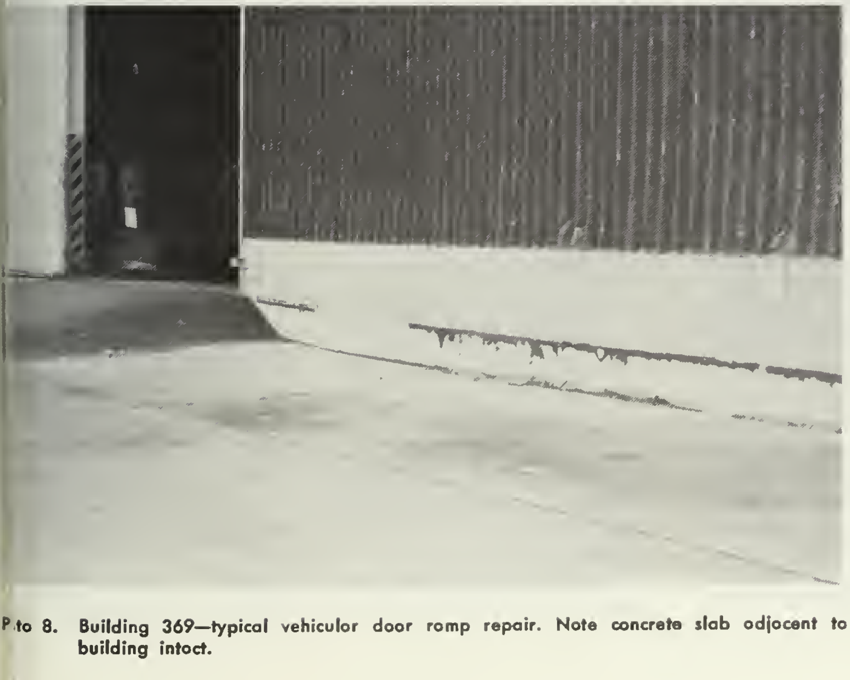


Photo 8. Building 369—typical vehicular door ramp repair. Note concrete slab adjacent to building intact.

Photo 9. Building 369—typical repair with asphalt paving adjacent to building where "hung-up" concrete slab has been removed. Also note vehicular door ramp repairs.

CASE HISTORY 4. CITY OF OAKLAND MUNICIPAL AUDITORIUM

By CHARLES H. LEE, Consulting Engineer

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84 Table 1. Settlement record, Oakland Auditorium—1914 to 1946.
85 Table 2. Test boring records, Oakland Auditorium—1934.

Piles for support of this building were driven in 1912 and the erection of the superstructure was completed in 1914. The building is 200 by 400 feet in plan and is located entirely within the soft bay mud area adjacent to Lake Merritt outlet. The mud beneath the building is 25 to 46 feet in thickness. It is underlain by alternating thin beds of blue clay, sand and gravel of aggregate thickness of 26 to 46 feet. Below this is a hard marly blue clay designated "good soil" by the engineer who inspected test borings drilled in 1934 after serious differential settlement had occurred.

The walls and roof of the building are supported on pile clusters placed beneath column footings, each cluster consisting of 12 to 24 piles spaced on 3-foot centers. The roof is supported by steel frame arches, three-hinged, pin connected with a span of 200 feet and spring line at level of top balcony seats. The arches are supported by the steel wall frame. The main floor is supported by piles 3 feet apart in pairs spaced on 17-foot centers in both directions. All piles were driven to 20 tons capacity as determined by the Engineering News formula, in common use at that time. No test borings were made.

Initial elevations were, top of piles + 2.5 feet, top of concrete capping for columns + 5.25 feet, and for floor supports + 7.5 feet, and main floor level + 19 feet.[1] Original average ground surface was + 9.5 feet (+ 11.3 feet MSL) but was excavated to + 2.0 feet around piles in order to make cutoffs and place concrete cappings. This permitted tidal waters to enter the area beneath the floor at high tide.

Soon after completion a horse show was held in the Auditorium and to protect the floor one foot of soil was spread. After the close of the fair this was dropped through trap doors onto the irregular mud surface below. Soon thereafter, dredger sand was placed beneath and around the building raising the general surface elevation beneath the building to at least + 9.5 feet, and outside several feet higher.

Soon after filling, serious settlement of the building occurred with spreading of the walls, necessitating emergency repairs which included the stringing of cables across the building to stabilize the arches. Very active settlement continued for several years, after which it gradually decreased but was still appreciable in 1926. Total settlements in 1946 varied from 0.29 foot at the southwest corner to 1.21 feet at the southeast. Settlement of the fill beneath the building exceeded 3 feet.

Table 1. Settlement record, Oakland Auditorium—1914 to 1946

Pin No.	Location	Total settlement in feet, 1914 to 1924	1926	1940	1946
1	S.W. Cor.	0.25	0.26	0.28	0.29
2	N.W. Cor.	0.40	0.42	0.46	0.46
5	166 ft. from N.W. Cor.	0.58	0.62	0.71	0.73
6	184 ft. from S.W. Cor.	0.50	0.55	0.78	0.81
8	98 ft. from N.E. Cor.	0.74	0.80	0.89	0.90
10	N.E. Cor.	0.85	0.92	1.09	1.10
11	S.E. Cor.	0.85	0.94	1.19	1.21
13	96 ft. from S.E. Cor.	0.85	0.94	1.16	1.19

As indicated by the data in Table 1, settlement was least at the west end of the building and greatest at the east end with appreciable differences between corners at the two ends.

---

[1] All elevations referred to Oakland City Base which is 1.77 feet above Mean Sea Level.

Table 2. Test boring records, Oakland Auditorium—1934.

Test boring number	Penetration record-feet						
	Fill	Mud	Gravel and clay	Hard marly blue clay	Total	Top of hard marly blue clay	Tips of adjacent piles as driven
3 A[1]	22	25	25	14	86	−57	−61 to −66
2 A[2]	20	45	21	14	100	−73	−59 to −72
4 A[3]	14	47	11	23	95	−57	−61 to −65
1 A[4]	16	36	42	35	129	−81	−61 to −68
6 A[5]	17	48	30	54	149	−82	−63 to −68
5 A[6]	20	51	15	11	97	−72	−61 to −64

[1] Center west end of auditorium.
[2] Center north side.
[3] Center south side.
[4] N.E. corner.
[5] Center east end.
[6] Near S.E. corner.

In 1934 six test borings were drilled around the periphery of the auditorium, one at the west end, two at the center and three at the east end. The logs of these test borings revealed the cause of the unequal settlement of the building, namely, that in the portion of the building where least settlement occurred (west end) the tips of piles penetrated the "good soil" (hard marly blue clay), while in the remainder of the building piles "took up" in the gravel and clay strata. Thus where settlement was least the piles had end bearing in the "good soil", in addition to friction in the overlying gravel and clay, while in the remainder of the building there was friction only. The following table presents details of the test boring logs and original elevations of adjacent pile tips.

The weight of the fill material placed under the building after completion greatly exceeded that of the mud excavated during construction and apparently this added weight was transferred in part by skin friction to the piles. Those piles at the west end of the building which penetrated the "good soil" experienced relatively minor settlement, but others which penetrated only the gravel and clay deposits, especially those at the east end of the building, experienced settlement three to four times as great as those at the west end. Not only were relative settlements great but total settlements were in terms of feet rather than in fractions of inches which are ordinarily considered tolerable. The arch type of construction which supported the roof was particularly vulnerable to excessive differential settlement.

Data presented above is from the report to the City Engineer, City of Oakland entitled "Report on Foundation Conditions at Proposed Development of 12th Street at Lake Merritt" dated February 13, 1947 by Charles H. Lee, Consulting Engineer, prepared under authority of Ordinance Nos. 2231 C.M.S. July 1, 1946, 2322 C.M.S. December 12, 1946, and C.M.S. 2354 February 3, 1947 of the City Council, City of Oakland.

SEISMIC PROBLEMS IN THE USE OF FILLS IN SAN FRANCISCO BAY

By H. BOLTON SEED, Professor of Civil Engineering
University of California, Berkeley

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SEISMIC PROBLEMS IN THE USE OF FILLS IN SAN FRANCISCO BAY

By H. Bolton Seed

INTRODUCTION

The probable behavior during earthquakes of filled areas overlying deposits of bay mud along the shoreline of San Francisco Bay has been the subject of much argument in recent years. Points of view have ranged from the position that fills overlying bay mud constitute a major hazard to life and property to, at the other extreme, claims that the presence of the bay mud is advantageous to the seismic behavior of the fill and the structures which it supports. In view of this wide range of opinions and the fact that the bay area lies in a zone of high seismic activity, it is important to analyze the situation to develop a clear picture of the possible seismic hazards associated with bay fill developments.

SEISMIC EFFECTS

The main seismic effects associated with earthquakes are:

Surface Faulting or Displacement

Earthquakes are caused by the sudden displacement of the earth along faults with a consequent release of stored strain energy. In larger earthquakes, the fault slippage will often extend to the ground surface where it is manifested by sudden and abrupt relative ground displacements. These displacements may be primarily horizontal as in the case of the San Francisco earthquake of 1906 where the maximum horizontal displacement was about 20 feet, or they may be primarily vertical as in the case of the Hebgen Lake, Montana, earthquake of 1959, for which the vertical displacement was about 20 feet. They may also produce a combination of horizontal and vertical displacements.

From an engineering point of view, no structure can be designed to withstand, without some damage, a large displacement which may occur along an underlying fault. A number of housing developments in California are located on major faults, and damage to houses and other buildings in the event of fault slippage in these areas is inevitable.

Fortunately, it does not appear that any important active faults underlie the bay itself except where the Hayward fault extends across San Pablo Bay. Thus, except in this area, no major problems of damage due to surface faulting are likely to develop in connection with bay fills.

Change in Elevation of Ground

Associated with vertical fault slippage in a major earthquake there is often an overall change in elevation of the ground level (tectonic movements) extending for several hundred miles from the fault itself. These movements, which may be up or down depending on the direction of movement of the fault, are so gradual in form that they are of little significance except along the coast line, where the relative movement of the land to water level may lead to flooding (in cases where the land moves downwards) or disruption of shoreline facilities and operations (in cases where the land moves upwards). In the Alaska earthquake of 1964, for example, the uplift and subsidence accompanying the fault movement affected an area of at least 34,000 square miles. East of the fault zone the land levels were raised as much as 38 feet, while to the west the general land level subsided as much as 7.5 feet. Thus coastal towns such as Cordova were raised about 6 feet relative to mean sea level, while Whittier and Seward subsided several feet.

As noted in the accompanying report by H. Goldman, recent historical displacements along the major faults in the bay area have been predominantly horizontal. However, geologic evidence indicates that older movements have involved significant vertical displacements. Thus it would seem prudent to consider the possibility of one or two feet of vertical displacement of the land surface adjacent to the bay simply as a result of the regional ground movements associated with major fault displacements.

Ground Shaking

The release of built-up strain energy accompanying slippage along a fault leads to severe ground vibrations which in general diminish in intensity with increasing distance from the fault. However, within about 15 miles of a major fault like the San Andreas, during a major earthquake ground vibrations on similar geologic formations are not likely to vary appreciably; beyond this distance they will decrease progressively, but they are still likely to be felt as much as 100 or more miles away.

Apart from the general decrease in vibration intensity with distance from the fault, local variations in shaking intensity are likely to occur due to variations in soil conditions. In addition, the shaking may have other detrimental effects on certain types of soils. Some of these other effects are discussed in the following section.

SOIL STABILITY PROBLEMS CAUSED BY EARTHQUAKES

Settlement of Cohesionless Soils

It has long been recognized that vibration is an effective means of compacting cohesionless soils, that is, soils containing no significant clay component. Thus it is not surprising that the ground vibrations caused by earthquakes often lead to compaction of cohesionless soil deposits and associated settlement of the ground surface.

A quantitative measure of ground settlement of this type was provided by the behavior of a well casing at Homer during the Alaska earthquake (1964). The casing had been installed to firm rock before the earthquake and projected about one foot above the ground surface. Following the earthquake the casing projected some 3.5 feet above the ground surface, indicating a decrease in thickness of the soil layer of about 2.5 feet. Tectonic movements caused the underlying rock surface to be lowered by 2 feet and this, together with the 2.5 feet of settlement caused by soil compaction, resulted in a total settlement of the ground surface of 4.5 feet.

A similar combination of effects in the Portage area of Alaska (1964) led to the town being inaccessible during periods of high tide. Here the combination of about 5 feet settlement of the rock due to tectonic movements together with 4 to 5 feet of settlement due to compaction of the overlying soil, led to a ground surface settlement of 8 or 9 feet. As a result, the highway from Anchorage to Portage, which was constructed on an embankment fill along the coastline, was submerged during periods of high tide, and the township had to move to a new location.

Similar problems of flooding and inundation of land due to settlement of cohesionless soil by compaction also occurred in the Chilean earthquake (1960) and the Niigata, Japan, earthquake (1964).

Ground settlements due to compaction may lead to differential settlement of engineering structures—a phenomenon which is particularly well illustrated by the performance of bridge abutments. Often an abutment is supported on firm materials or on a pile foundation and undergoes relatively small settlements compared with the backfill material for the abutment, which rests directly on the ground surface and settles due to compaction of the soil on which it rests. In the case of a bridge near Portage, Alaska, the piers and abutments were supported on piles, but the abutment piles extended to a greater depth than those under the piers. As a result the piers settled more than the abutment (due to the greater depth of soil below them compacted by the earthquake) and the bridge deck was left spanning across the stream bed, unsupported by the piers on which it had previously rested. Backfill settlement relative to the abutment left the railroad tracks unsupported behind the bridge abutment.

Soil Liquefaction—Level Ground

If loose or medium dense saturated cohesionless materials are subjected to earthquake ground vibrations the resulting tendency to compact must be accompanied by an increase in water pressure in the soil and a resulting movement of water from the voids. Water is thus caused to flow upwards to the ground surface where it emerges in the form of mud-spouts or sand boils. The development of high water pressures in soils due to ground vibrations and the resulting upward flow of water will often turn a sand into a "quick" or liquefied condition (it may be recalled that quicksand is a condition induced by a sufficiently large upward flow of water through a sand).

Liquefaction of saturated sands is a rather common phenomenon during earthquakes but nowhere has i been more dramatically illustrated than in the town of Niigata, Japan, during the earthquake of June, 1964 The epicenter of the earthquake (magnitude abou 7.3) was located some 35 miles from Niigata bu nevertheless the earthquake induced extensive liquefaction of the sand deposits in the low-lying areas o the town.

As liquefaction developed over extensive areas, automobiles, structures and other objects gradually settled into the resulting quicksand, and several light-weigh buried structures floated to the surface. A sewag treatment tank which was originally buried with it base some 15 feet below the ground surface, projected about 10 feet above the surface when the ground motions subsided.

Many structures settled more than three feet and the settlement was often accompanied by severe tilting. Undoubtedly the most dramatic case of settlin and tilting was that experienced by the apartment buildings at Kawagishi-cho (figure 1). One of these buildings tilted through an angle of 80° and the occupants were able to evacuate, after the motion stopped by walking down the face of the building. Neighboring buildings suffered varying lesser degrees of tilting

The airport building settled three feet and extensiv liquefaction occurred in the surrounding area, causin severe damage to runways and other facilities.

Similar effects to those described above occurre at Puerto Montt during the Chilean earthquake of Ma 1960. The southerly part of the city is located on low flat land adjacent to the waterfront. Along the waterfront itself, structures are constructed on a granula fill, placed in some areas by dumping and in others b hydraulic methods. Both types of fill liquefied extensively during the earthquake. Similar liquefaction o sands and silts underlying level ground is reporte to have occurred at Valdivia and other locations.

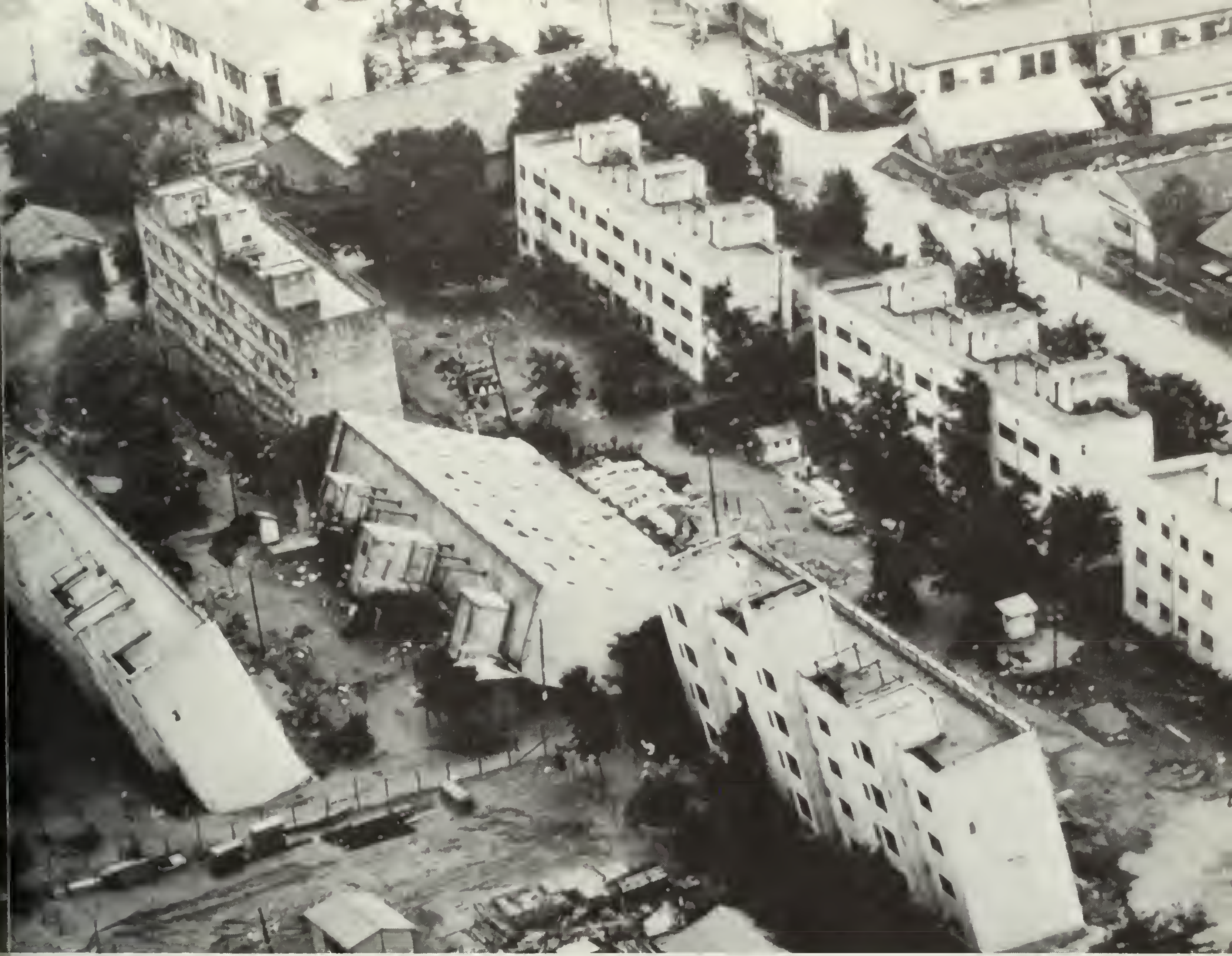

Figure 1. Tilting of apartment buildings at Kawagishi-cho, Niigata.

Soil Liquefaction in Sloping Ground—Flow Slides

If liquefaction occurs in or under a sloping soil mass, the entire mass, together possibly with adjacent liquefied deposits, will flow or move laterally to the unsupported side in a phenomenon termed a flow slide. Such slides also develop in loose saturated cohesionless materials during earthquakes and are reported in Chile (1960), Alaska (1964), and Niigata (1964).

Flow slides developed in loose fill along the waterfront at Puerto Montt (described above) as evidenced by the lateral separation of about 9 feet which occurred between two halves of a building, one part of which rested on liquefied soil. A similar phenomenon occurred at a school building constructed near the river bank at Niigata, and lateral spreading of ground oward surface depressions was observed at a number of locations in the Alaska earthquake.

More extensive flow slides also occurred during the Alaska earthquake at Valdez, Seward, Kenai Lake and various other locations. At Valdez and Seward, alhough large parts of the slide masses were under water, extensive sections of the waterfront (about ,000 feet of coastline at Seward) were carried away by the flow slides.

Probably the largest flow slides in recorded history ccurred in Kansu Province, China, during an earthquake in 1920. Again the soil involved was a loose loess-type granular deposit. Large flow slides in this material occurred over an area 100 square miles in extent. Cities were buried and farmsteads were carried away to be set down in new locations once the movements stopped. In one area a section of a roadway, with its border of poplar trees, was transported and set down one mile from its original location. It is reported that some 200,000 people were killed during these enormous movements.

However, flow slides may also be induced on a smaller scale by relatively small earthquakes, as evidenced by the slides which occurred along the highway bordering Lake Merced in San Francisco during an earthquake of magnitude 5.3 in 1957. The highway was constructed on several feet of fill resting on a saturated loose sand deposit. Liquefaction of the loose sand resulted in sections of the highway being carried some distance into the lake, illustrating the large lateral translations characteristics of flow slides.

Waterfront Bulkhead Failures Due to Backfill Liquefaction

Since liquefaction develops most frequently in loose saturated cohesionless soils, it is not surprising that many instances of waterfront bulkhead failures during earthquakes have been reported due to liquefaction of the backfill. Such bulkheads usually consist of sheet pile walls or reinforced concrete walls which are placed in water along a coastline and then backfilled to create a dock or pier. Backfill materials are usually sand or sand and gravel. Since it is difficult to compact the backfill below the water level, it often consists of a loose saturated sandy material which is extremely vulnerable to liquefaction. The walls themselves are usually designed to withstand the relatively low pressures applied to them by the non-liquefied sand backfill. Thus if the backfill liquefies during an earthquake, the increase in lateral pressure accompanying liquefaction may well cause the wall to be pushed outwards. Quay walls and bulkheads in dock areas have often suffered major damage during earthquakes due to these effects.

Extensive failures of quay walls and bulkheads as a result of backfill liquefaction occurred at Puerto Montt (1960) and Niigata (1964). At Puerto Montt, during the Chilean earthquake, the land along the coast flowed into the sea over a length of 2,000 feet, carrying the retaining structures with it. Quay walls, consisting typically of 16-foot reinforced concrete sections constructed on about 35-foot high soil-filled concrete caissons, overturned completely over a length of about 900 feet; over another section 700 feet long, the reinforced concrete upper section overturned completely while the lower caisson section tilted outwards. The fill behind the walls disappeared into the bay. Anchored sheet pile bulkheads along the waterfront were pushed outwards up to three feet over a length of about 1,250 feet, with resulting settlement and distortion of the railway tracks behind them.

Similar damage occurred to the sheet pile bulkheads with loose sand backfills in the harbor area of Niigata. Extensive sections moved outwards under the increased backfill pressures developed by the backfills during the earthquake, with resulting damage to structures behind them due to settlement and lateral translation of the foundations.

Extensive failures of walls involved in flow slides at Valdez and Seward in the Alaska earthquake have already been described.

Slides Caused by Liquefaction of Thin Sand Layers

Loose saturated silts and sands often occur as essentially horizontal, thin seams or layers underlying firmer materials. In such cases liquefaction of the sand induced by earthquake ground motions may cause an overlying sloping soil mass to slide laterally along the liquefied layer at its base. When this happens, a zone of soil at the back end of the sliding mass sinks into the vacant space formed as the mass translates, resulting in a depressed zone known as a graben. During motions of this type, structures on the main slide mass are translated laterally, often without significant damage, but buildings in the graben area are subjected to such large differential settlements that they are often completely destroyed. Furthermore, buildings near the toe of the slide area, where the soil pushes outwards, may be heaved upwards or pushed over by the lateral thrust.

Several slides of this type occurred in Anchorage during the Alaska earthquake of 1964. The L-Street slide area involved a soil mass about 4,000 feet in length parallel to the bluffs and extending back from the toe as much as 1,200 feet. The graben, which formed behind the slide mass varied in width from 100 to 250 feet and in depth from about 7 to 10 feet.

Similar grabens, causing extensive destruction of buildings, developed in the 4th Avenue and Government Hill slide areas.

Landslides in Clay Soils

Major slide movements during earthquakes have also occurred in clay deposits. There is no evidence that clays liquefy during earthquakes in the sense that an extensive zone of clay soil is converted into a fluid condition. However, some types of clay may lose a proportion of their strength and the additional forces induced by an earthquake may cause slope failures with resulting slumping. In addition, clay deposits often contain sand lenses and liquefaction of these lenses may lead to significant landslide development.

One of the largest slides in a clay deposit was that which occurred along the coastline of the Turnagain Heights area of Anchorage during the Alaska earthquake of 1964. The coastline in this area was marked by bluffs some 70 feet high and sloping at about 1½:1 down to the bay. The slide extended about 8,500 feet from east to west along the bluff line and retrogressed inland an average distance of about 900 feet. The total area involved in the slide was thus about 130 acres.

Within the slide area the original ground surface was completely devastated by displacements which broke up the ground into a complex system of ridges and depressions. In the depressed areas between the ridges the ground dropped an average of 35 feet during the sliding. The east end of the slide area had been developed for residential purposes and about 75 houses in this section of the area were destroyed. A study conducted by the Engineering Geology Evaluation Group in Anchorage revealed that some of the houses moved laterally as much as 500 to 600 feet during the sliding.

It appears that liquefaction of sand lenses within the clay played a significant role in the development of the Turnagain Heights slide.

Three somewhat similar slides occurred on the San Pedro River near Lake Rinehue during the Chilean earthquake of 1960. The largest of these involved about 30 million cubic yards of material which moved vertically about 60 feet and laterally about 1,000 feet during the sliding. The other two slides involved about 6 million and 2 million cubic yards of soil respectively. The slides blocked the course of the river and raised the water level in the lake by 80 feet, thereby endangering the city of Valdivia some 35 miles to the west. It is of interest to note that a similar effect occurred when an adjacent area slid into the river during an earthquake in 1575.

Sloping Fills on Firm Foundations

The effect of earthquakes on banks of well-compacted fill constructed on firm foundations in which no significant increases in pore-water pressure develop during the earthquake, is characteristically a slumping of the fill resulting in settlements varying from a fraction of an inch to several feet, depending on the slope. However, such slumping is negligible in gently sloping sections.

Sloping Fills on Weak Foundations

In contrast to the behavior of sloping fills on firm foundations, many cases have been reported in which embankments on weak foundations have either failed completely or slumped severely with associated longitudinal cracking.

An excellent example is provided by the behavior of an embankment section of the north-south highway between Puerto Varas and Puerto Montt, Chile. A four-foot high fill, about 1,500 feet long, was constructed where the highway passed over a swampy area of ground and surfaced with an eight-inch thick concrete pavement. During the earthquake of 1960 the embankment collapsed completely, and the concrete pavement slabs were deposited at the level of the swamp. The highway was undamaged before entering and after leaving the swamp area.

Similar behavior was observed in a dike, built to a height of seven feet along the bank of a canal in northern Mexico, which sank completely into the underlying soft foundation soil, and the complete collapse of the side fill of a highway bridge approach ramp during the El Centro earthquake of 1940.

Such failures seem to be characterized by lateral spreading of the soil at the base of the fill underlying the slopes. Where movement due to spreading is less severe, the resulting effect is usually severe longitudinal cracking of the fill rather than a complete failure. This type of behavior developed along many sections of highway embankments in the Alaska and Niigata earthquakes of 1964.

Slumping and settlement of embankments in the San Francisco Bay area during the 1906 earthquake were also described by the Committee on the Effect of the Earthquake on Railroad Structures as follows:

"Embankments across marshes or with soft strata underlying them settled more or less. In some cases the settlement was vertical; in other cases there was considerable horizontal with the vertical movement. . . . At one point on the marsh between Benicia and Suisun, on the Southern Pacific, the settlement was 11 feet; at another point 5 feet. These were nearly vertical. . . . On the North Shore, about 2 miles north of Point Reyes, the road originally had been constructed with pile trestles across several arms of Tomales Bay; these trestles had been filled about 15 years ago, the roadbed being about 8 feet above ordinary high tide. Two of these embankments, 2,200 feet and 900 feet long, respectively, sank until the water at high tide washed over the rails. . . . Settlements of this character, and of more or less severity, were common on all railroads operating about the Bay of San Francisco and its tributaries."

While some of these settlements were undoubtedly due to compaction of loose fill material, it would appear that displacement of the underlying soft deposits probably contributed to the overall movements in some cases.

One of the most dramatic examples of the failure of an entire fill resulting from loss of strength of the foundation soil during an earthquake is the failure of the Sheffield Dam during an earthquake near Santa Barbara in 1926. During the earthquake observers report that a section of the dam, about 300 feet long, slid downstream resulting in a complete loss of water from the reservoir.

Retaining Walls and Bridge Abutments

The tendency for earth fill slopes to slide downhill during earthquakes necessarily results in increased pressures on retaining walls. Such pressures often result in wall displacements.

Similar displacements occur in bridge abutments, which serve the dual purpose of supporting the bridge deck and retaining the approach fill. Inward movements of bridge abutments during earthquakes causing a distortion of the bridge structure have often been reported.

INFLUENCE OF GROUND SHAKING ON BAY FILLS AND THEIR FOUNDATIONS

The preceding section described the effects that earthquake ground shaking may have on soils. It should not be construed from this that such effects will necessarily develop during earthquakes; rather it is the responsibility of the engineer to minimize the possibility of these effects developing, and recognition of the types of problems involved is an important first step in this direction. Engineering solutions can usually be devised to reduce the hazards from all types of soil instability problems to acceptable proportions, if the hazard is recognized at a sufficiently early stage. For example, the problem of settlement of sands during earthquakes can be avoided by compacting the sand to a sufficiently dense condition during construction, the possibility of sliding in earth banks can often be minimized by flattening slopes, or the possibility of liquefaction of fills can be eliminated either by an appropriate choice of construction materials or by adequate compaction.

Thus the possible effects of earthquakes on soils are determined in large measure by the configuration of the ground surface and the nature of the soils involved. It follows that the effects of earthquakes on bay fills will be largely determined by (1) the nature of the foundation materials on which the fills are placed and (2) the nature and configuration of the fills themselves. Let us look now at the nature of these materials as they are likely to exist in the bay area.

Foundation Soils

No matter how good a fill may be, shear failure and lateral displacement of the foundation soils on which it rests will inevitably lead to instability problems. Thus, in the design of fills careful consideration must always be given to the foundation soils. Fills placed along the shoreline of the bay are likely to be supported directly on either (1) a layer of San

Francisco Bay mud or (2) deposits of sand (see report by H. Goldman). The nature of these deposits and the possible effects of earthquake ground motions on them may be summarized as follows:

San Francisco Bay Mud

San Francisco Bay mud is a soft, slightly organic silty clay containing occasional thin seams or lenses of silt and sand. Its high water content and low preconsolidation pressure cause it to undergo substantial long term settlement under sustained loads and its low shear strength may lead to plastic flow in zones of substantial shear stress, such as occur at the edges of steep fill slopes. The problems associated with these characteristics have been discussed by Lee and Praszker.

Investigations of the behavior of the clay under seismic loading conditions indicate that:

a. Under level ground conditions, there is no evidence to indicate that the clay portion of the deposit would settle or liquefy during an earthquake.

b. The sand seams in the clay may liquefy during strong ground motions, but this is unlikely to have any serious effects unless the liquefied layer underlies or is immediately adjacent to a sloping surface and is located at a depth permitting sliding to occur. Otherwise the only effect would be a very small settlement as the pore pressures in the liquefied seams dissipated after the earthquake.

c. Slopes and adjacent land composed of or underlain by the clay may slide during an earthquake due to the additional forces developed in them by the earthquake, unless they are of sufficiently conservative design.

d. The deformable nature of San Francisco Bay mud will cause any deposit containing a substantial thickness of the clay to vibrate with a relatively low frequency of vibration. As a result, ground surface motions would have lower frequencies than those on adjacent areas not underlain by bay mud deposits. The influence of this effect will be discussed in a later section of this report.

Sand Deposits

Sand deposits in the bay are likely to be loose to medium dense and thus under earthquake loading conditions they are potentially vulnerable to:

a. Settlement due to soil compaction.

b. Liquefaction with resulting settlements of overlying materials and structures such as occurred at Niigata, Japan (1964).

c. Liquefaction and resulting flow slides such as occurred at Valdez and Seward during the Alaska earthquake of 1964.

The extent to which these effects might develop will depend on the in-place density of the deposit, the geometry of each particular case, and the intensity and duration of the earthquake. However, the possibility of these effects should be considered in cases where fills might be placed on sand deposits.

Fill Materials

The main types of fill materials likely to be used in the bay area may be separated into three main classes as follows:

Uncompacted Dumped Fill of All Types of Soil

Uncompacted dumped fills are naturally loose and form poor foundations for almost all types of structures. With time, however, they may densify sufficiently to support houses or other lightweight buildings. Under the influence of earthquake-induced vibrations, these materials:

a. Are likely to settle substantially, leading to appreciable differential settlements of buildings supported on them.

b. May liquefy if the fill is saturated and sandy in nature.

c. May undergo some degree of lateral movement due to differential movements within the fill.

d. May develop slides or slumping in sloping boundaries, especially where the fill is saturated.

The above effects are likely to be far more deleterious in their effects on structures than are any undesirable modifications of the ground motions, due to the nature of the fill itself.

This type of behavior is best illustrated by the behavior of old bay fills in the 1906 San Francisco earthquake. It is apparent from the accompanying report by Mr. Steinbrugge that these fills were loosely dumped and their behavior is described by H. O Wood in the Report of the State Earthquake Investigation Commission as follows:

> "About the Ferry Building, at the foot of Market Street, is a district of 'made' land in which high intensity was manifested . . . In spots the streets sank bodily as much as 2 feet, probably more. Accompanying this depression, concrete basement floors were broken and arched, as if to compensate for it. The surface of the ground was deformed into waves and small open fissures were formed, especially close to the wharves. Buildings on the water side, along East Street, generally slumped seaward as much as 2 feet. The damage was greatest close to the water's edge, growing less as the solid land was approached . . . The shaking caused the materials used in filling to settle together and occupy less space so that the surface over the whole area was lowered by amounts varying from a few inches to 3 feet or more. This is clearly seen in the change of street levels along the margin of the solid ground, where the car rails are bent downwards in little monoclines. Occasionally a structure with a relatively good foundation remains at its former level, with the whole neighborhood depressed about it."

Some indication that the intensity of shaking in th filled areas was not in itself responsible for the extensive damage which developed is provided by th statement:

> "Buildings erected upon good foundations withstood the ordeal well, even when the streets around them were depressed and fissured."

Thus it would seem that buildings which did no suffer differential settlement or lateral displacemen were able to withstand the shaking which develope in the filled areas approximately as well as simila buildings in non-filled areas. Since the foundations o a building underlain by clay soils have virtually n influence on the intensity of shaking at a building sit t would thus appear that the intensity of shaking in the filled areas was not enormously different from that n the non-filled areas.

It is important to note that uncompacted dumped ills bear little resemblance to modern, well-compacted, rolled earth fills and that the behavior of the fills in he 1906 earthquake cannot therefore serve as a guide o the behavior of well-compacted fills in future arthquakes.

Hydraulic Sand Fills

Hydraulic sand fills have been used extensively in he Bay Area. If the hydraulically deposited soil is not ubsequently compacted, it is often in a loose to medium dense condition and thus during earthquakes potentially subject to:

a. Some degree of settlement induced by the ground ibrations; however the settlement will be appreciably ss than that of loosely dumped fills.

b. Possible liquefaction depending on the density chieved during placement, the depth of the water ble and the intensity of the ground motions.

c. Flow slides if the sand should liquefy adjacent to nsupported sloping boundaries at the edges of the fill.

Extensive liquefaction developed in hydraulic sand lls at Puerto Montt during the Chilean earthquake of 960, indicating the potential dangers associated with is type of material. On the other hand, the experinces at Niigata, Japan, in the earthquake of 1964 show onclusively that loose to medium dense sands will not ecessarily liquefy, that small changes in depth of ater table and severity of shaking can have significant fects on liquefaction potential, and that a careful praisal of the significant details involved in each uation is required before an evaluation of the possible dangers of liquefaction developing can be made.

Thus while hydraulic sand fills are potentially vulnerable to settlement, liquefaction and flow slides, it ould be possible, under favorable conditions of material characteristics, soil profile, and water table elevation, to utilize these materials for bay fills without trimental consequences. It cannot be stressed too ghly, however, that the best possible appraisal of tese possibilities should be made whenever hydraulic f s are utilized.

ell Compacted Fills of Select Material

During the past twenty years significant progress has been made in the development of earth moving and compacting equipment and in the development of a better understanding and appreciation of the characteristics of compacted soils. Thus most fills constructed in this period under engineering supervision have been composed of materials suitable for compaction to a dense, firm condition and appropriate measures have been taken to ensure that they have been placed in such condition in the field.

Soil fills placed and compacted in this way are much sounder materials than many natural deposits and apart from possible slope stability problems when the fill is saturated, they present no serious hazards with regard to their behavior during earthquakes. Well-compacted select fill materials are not likely themselves to settle appreciably, undergo liquefaction, or, if slopes are conservatively designed, be involved in landslides. However, they will of course be affected if these phenomena should develop in the underlying foundation soils.

Thus if well-compacted fills are placed with conservative side slopes on firm foundation materials, they should present no special hazards, other than ground shaking, during earthquakes. Some evidence of this is provided in Mr. Steinbrugge's report of the behavior of fills in the Westlake Palisades Tract during the San Francisco earthquake of March 22, 1957:

> "These fills were not over bay muds or other compressible bay soils, but certainly their performance shows that man-made fills, per se, can be quite satisfactory in moderate earthquakes. In contrast, damage at the San Francisco International Airport about 10 miles southeast of the epicenter was attributed to both differential settlements and horizontal vibratory forces."

Although the 1957 San Francisco earthquake was not nearly so strong as the 1906 shock, the performance of the fills is not indicative of any major cause for concern.

It should perhaps be noted that even well-compacted fills on firm foundations may, if they are deep, undergo settlements of several inches during a major earthquake. However, many natural soils would probably behave in a similar manner.

During earthquakes the main dangers with well-compacted fills are likely to be slope instability at the edges of the fills, due either to shear displacements in saturated fill materials or in foundation soils, and failure to adequately design structures for the nature of ground surface shaking that may develop. This would indicate a need for special studies and considerable conservatism in the selection of slopes and a careful consideration of the nature of ground surface motions to be expected in filled areas overlying bay deposits. This latter question will be discussed in some detail in the following section of this report.

NATURE OF GROUND MOTIONS FOR WELL-COMPACTED FILLS PLACED ON BAY MUD

Frequency of Ground Vibrations

The character of the shaking developed at the ground surface during an earthquake is influenced to a considerable degree by the nature of the entire soil deposit overlying relatively sound basement materials at the site. Both field instrumental data and scientific analyses show that in general the softer the nature of the underlying soil deposits, the lower will be the predominant frequency of vibration of the ground surface motions.

The effect of the ground motions on buildings and structures depends not only on the characteristics of the ground motion but also on the vibration characteristics of the buildings themselves. The vibration characteristics of a building are conveniently expressed in terms of its fundamental frequency of vibration—that is the predominant frequency with which the building would vibrate if it were pulled sideways at the top and then suddenly released, without simultaneously being subjected to any other source of vibration. Thus, for example, a stiff single-story building would typically have a fundamental frequency of vibration of about 20 cycles per second, a 10-story building might have a fundamental frequency of about 1 cycle per second and a 40-story building might have a fundamental frequency of about 0.25 cycle per second.

If a building is subjected at its base to a series of vibrations having the same frequency as the natural frequency of the building, then large amplitude motions and large forces develop in the building. On the other hand if the same building is subjected to base vibrations having a frequency very different from the natural frequency of the building, comparatively small effects will be induced in the building.

Thus in order to minimize the effects of ground shaking on buildings, it is desirable to develop as much differentiation as possible between the fundamental frequency of the building and the predominant frequency of the ground motions. For buildings constructed on fills over bay mud, for which the predominant frequency of ground surface motions will be relatively low, vibration effects are likely to be smallest for buildings with high frequency characteristics, such as single-story structures, and largest for buildings with low frequency characteristics such as multi-story structures.

This does not mean that multi-story buildings cannot be constructed at sites underlain by soft soils such as bay mud. However, it must be recognized that such structures will be subjected to more severe shaking than smaller structures and they must be designed accordingly because of the increased seismic risk.

Mr. Steinbrugge describes an excellent illustration of the above effects in Mexico City, much of which is underlain by a deep bed of clay:

> "A classic example of selective damage to tall buildings (i.e., long period structures) compared to rigid one- and two-story buildings occurred in the July 20, 1957, Mexican earthquake. The earthquake was distant with its epicenter reported from 170 to 220 miles from Mexico City. Damage was pronounced to many of the tall reinforced concrete and steel-frame buildings in downtown Mexico City, and several collapsed. Small one- and two-story collapse-hazard buildings performed well."

Similar effects were also apparent in Anchorage during the Alaska earthquake of March 1964. This city is also underlain by a deep bed of soft clay which would result in long period (low frequency) ground surface motions. In areas where building damage was not related to landsliding, major damage occurred in multi-story buildings with adjacent single-story structures being virtually undamaged.

Thus there is much evidence to support Mr. Steinbrugge's conclusion :

> "In a general sense, the seismic risk to buildings on Bay fills from ground motions . . . is least for one-story light mass structures of no more than moderate floor areas."

Amplitude of Ground Vibrations

While there is reasonably general agreement on the effect of soft soil deposits on the predominant frequency of the ground surface vibrations, there is considerable disagreement on the effects of such soils o the amplitude or intensity of these vibrations.

There is a widespread tendency to believe that a earthquake ground motions are amplified and are man times larger for areas underlain by soft alluvial d posits than for adjacent areas underlain by rock; fu thermore, that the severity of the ground motic tends to increase with increasing depth of alluvi deposits. However, such concepts are probably a oversimplification of the actual conditions, and th merit close scrutiny before being used as the basis f conclusions.

Evidence supporting the argument that alluvial d posits cause considerable amplification of ground m tions comes primarily from two sources:

1. There is much instrumental evidence from a tual recordings that ground accelerations recorded alluvial deposits are many times larger than those adjacent rock areas. However, almost all of these re ords pertain to extremely small earthquakes and whi the result is undoubtedly true for micro-tremors, the is little justification for extrapolating the results earthquakes (possibly releasing 100,000 times mo energy) inducing strong ground motions. Duri strong ground motions, the laws controlling soil b havior are quite different from those applicable duri weak ground motions and a great deal more ener is absorbed by the soil. Thus, any amplification ground motions during large earthquakes would expected to be very much smaller than those observ during very small shocks.

It is worthy of note that there are no records fro zones of strong ground shaking in major earthquak to show the extent to which ground motions we amplified on soft soil deposits. This type of inform tion for major earthquakes is badly needed for scie tific, engineering, and planning purposes.

Analyses of soil response in Anchorage during t recent Alaska earthquake indicate that surface motio may have been only about 30 percent greater th those in the base rock, and similar studies for the 20 foot layer of sand at Niigata indicate an amplificati of about 50 percent.

2. A considerable amount of evidence shows th damage to buildings located on loose or soft soils considerably greater than that to adjacent buildings firmer materials and there is a tendency to attrib this to more violent shaking in the loose or soft s areas.

In his report on the relationship between grou conditions and damage in the 1906 San Francis earthquake, Wood concluded:

> "This investigation has clearly demonstrated that the amount of damage produced by the earthquake of April 18 in different parts of the city and county of San Francisco depended chiefly upon the geological character of the ground. Where the surface was solid rock the shock produced little damage; whereas upon made land great violence was manifested."

Wood also prepared a map showing variations intensity (see accompanying report by K. Ste brugge), which correlate broadly with geological c ditions. However as noted previously, a careful re ing of Wood's report indicates that buildings in fil areas which did not themselves settle were no more damaged than buildings in firm soil areas and thus it appears that the shaking intensity was not significantly more severe in the loose fill areas than in firm soil areas. In the light of this fact, the intensity referred to by Wood is presumably a damage intensity and other factors, such as ground settlement, and lateral displacement are likely to play as important a part in this categorization as shaking intensity.

Similar effects have been manifested in other earthquakes and while it is true that loose and soft soils have led to much greater damage levels than have firm soils and rocks, the reasons for this have not necessarily been simply the greater severity of shaking but also greater susceptibility of soft and loose soils to landsliding, lateral displacements, and differential settlements.

This is not to say that some amplification of ground vibrations has not occurred—but there is no reason to believe that it has been particularly large or anywhere near the order of magnitude observed for microtremors and very small earthquakes.

A final factor which must be considered in assessing the influence of soil conditions on the intensity of ground shaking is the recent availability of analytical procedures for evaluating this factor. While there have been limited opportunities to check the validity of these analyses, the results obtained have been in good agreement with field conditions in the few cases available for corroboration studies. Application of these techniques to typical fill over bay mud conditions indicates, for high intensity shaking, much smaller amplifications of adjacent rock motions that have previously been anticipated. Furthermore, the analyses indicate wide variations in intensity of surface shaking in the same general area, depending on the depth and character of the soils (bay mud or sand) underlying the fill.

In the light of all available information, it would ius appear that while some degree of amplification f ground motions for some areas underlain by soil eposits can certainly be anticipated, the amount will epend greatly on the depth and character of the deposit and the intensity of the ground motions themlves. For fills on deep layers of clay in the bay area, ie amplification of adjacent rock motions is not likely) exceed acceptable design limits for strong motions duced by major earthquakes. As a matter of fact, reater amplification of soil over adjacent rock motions might well be expected for many other natural il deposits in the bay area.

Probably the soil conditions most analogous, from a ground shaking point of view, to well-compacted fills overlying deep bay mud deposits, and for which seismic experience is available, are the soil deposits in Anchorage, Alaska. The soil profile in the Turnagain Heights area, well behind the coastal bluffs, is shown in figure 2. For purposes of comparison, the conditions at Treasure Island in San Francisco Bay are also shown in figure 2.

During the past 25 years, Anchorage has been subjected to a number of strong earthquakes as follows:

1. Magnitude 7.3 at an epicentral distance of about 35 miles (1943)
2. Magnitude 6.3 at an epicentral distance of about 50 miles (1951)
3. Magnitude 6.8 at an epicentral distance of about 60 miles (1954)
4. Magnitude 8.3 at an epicentral distance of about 70 miles (1964)

However, in areas where no landslide-related damage has occurred, there are no records of any substantial damage to residential structures or other lightweight short period buildings of one- or two-story construction.

While San Francisco Bay fills might be subjected to somewhat stronger ground motions than those listed above, the performance of the Anchorage soils is indicative of the fact that if settlement, liquefaction, and landsliding are avoided, ground shaking on filled areas will not necessarily lead to any appreciable damage to appropriately designed structures during strong earthquakes.

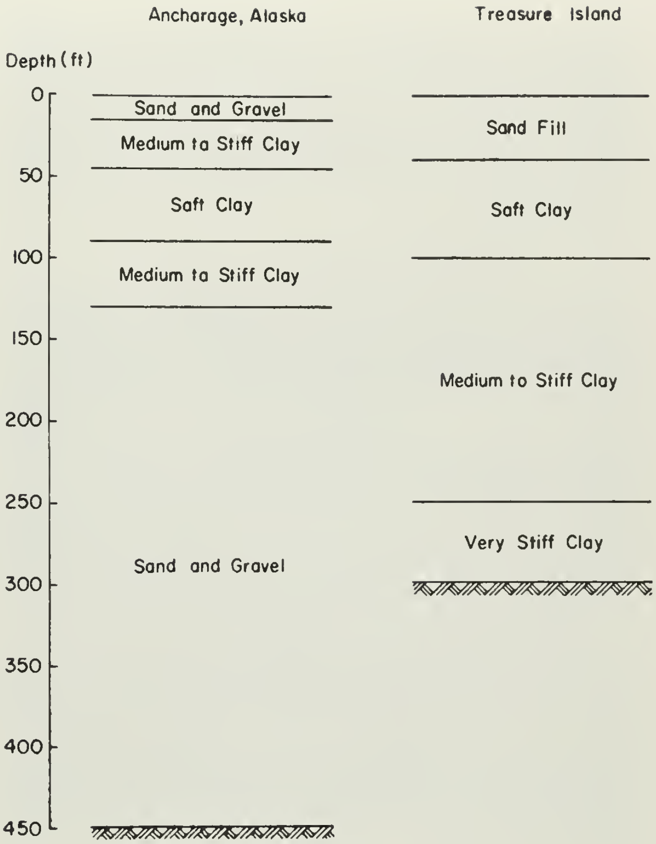

Figure 2. Typical soil profiles—Anchorage, Alaska, and Treasure Island.

EARTHQUAKE-RESISTANT DESIGN PRECAUTIONS IN CONNECTION WITH DEVELOPMENT OF BAY FILLS

In the light of the preceding discussion, the following design precautions should be noted in connection with the earthquake-resistance features of San Francisco Bay fills:

Recognition of Major Problems

Engineers, land developers and public authorities responsible for the construction of bay fills should be fully cognizant of the major problems (subsidence, settlement, landsliding, liquefaction, and ground shaking) which may develop and ensure that appropriate steps are taken to reduce the effects of these problems to acceptable limits.

Provision of Adequate Density in Fill

In order to ensure that no detrimental settlements will occur during earthquakes due to compaction of the fill itself, the fill should be placed during construction in a sufficiently dense condition that additional compaction by earthquake-induced ground motions will be minimal. This requirement is likely to be met automatically in well-compacted fills meeting the usual specifications for these materials (a degree of compaction of at least 90 percent on the modified AASHO compaction test). However, it may require special studies in the case of hydraulically deposited sand fills.

Investigation of Probable Settlement of Foundation Soils

In addition to ensuring that settlement of the fill itself will be minimal, an evaluation should also be made of the settlement likely to develop during an earthquake due to compaction of underlying foundation soils. Such settlements may vary from negligible settlements for clays to substantial amounts for loose cohesionless soils.

Investigation of Liquefaction Potential

In all fill projects involving loose or medium dense saturated silts or sands (either as fill itself or as a foundation for some other type of fill) a careful appraisal should be made of the possibility of liquefaction of the soil during earthquakes and the possible consequences of such liquefaction. For this purpose previous experience of soil liquefaction during earthquakes should be utilized to the fullest possible extent together with appropriate analytical techniques. It should be recognized that broad generalizations cannot be made in connection with liquefaction phenomena and that each case should be evaluated separately with due consideration being given to the maximum probable severity of earthquake ground motions, the depth of the water table and the density of the sand or silt deposit. In cases where liquefaction may develop, steps should be taken (e.g., use of greater depth of soil above water table, densification of soil) to eliminate this possibility.

Evaluation of Slide Potential

The best possible evaluation should be made of the possibility of landslides occurring due to (1) flow slides in saturated sands, (2) liquefaction of sand seams or lenses within clay deposits, (3) the driving forces exceeding the resistance of saturated fill and foundation soils, or (4) surface slumping of fill slopes. All slopes should be conservatively designed to reduce the risk and consequences of any slide development to acceptable limits.

Consideration of Probable Nature of Ground Surface Shaking

After ensuring that all major stability problems involving permanent displacements of the fill or foundation materials have been eliminated or reduced to tolerable proportions, consideration should be given to the probable nature of the ground shaking to which structures supported on the fill will be subjected. These motions will not be influenced significantly by foundation type (footings, piles, piers, etc.) but they will depend in large measure on the magnitude of the earthquake, the epicentral distance, and the nature of the entire soil deposit (including fill) overlying base rock at the site.

From a knowledge of the soil conditions and the probable magnitude and location of the design earthquake, it is possible to estimate a response spectrum characterizing the nature of the ground surface motions and their influence on buildings. This is particularly necessary in the case of fills over deep layers of bay mud since the response spectrum for these conditions is likely to differ appreciably from the average spectra used for other purposes. Buildings at the site should then be designed to safely withstand the maximum seismic forces indicated by the response spectrum in addition to the static stresses imposed by normal loads and any differential settlements. In general, since the predominant period of the ground motions is likely to be relatively high for areas underlain by thick layers of bay mud, seismic risks and seismic forces due to shaking are likely to be least for short period one-story structures of moderate floor area, and seismic forces will be large for multi-story structures.

Foundations

Since the soil underlying a building will be subjected to some degree of horizontal and vertical displacements during an earthquake, it is desirable to minimize differential displacements at the base of the structure as far as possible. Thus consideration should be given to the use of stiff mat or grid type foundations or where buildings are supported on piles, to the provision of stiff connections between individual footings. Such foundation systems would also help to minimize the stresses developed in the buildings by any differential settlements which develop due to static load effects.

With regard to the performance of pile foundations during earthquakes, there is good evidence to show that the use of piles in areas subjected to earthquake-induced settlements, can prevent major damage to the buildings. However, in the design of pile foundations, the following precautions should be observed:

1. Presently available evidence indicates that the presence of piles is not likely to affect significantly the nature of the ground shaking to which a building is subjected.

2. If piles are driven into granular soils in which settlement may be induced by the ground shaking, settlement and possibly differential settlement of the building foundation will result (see page 90).

3. If the soil around a pile settles during an earthquake, the pile will be subjected to an additional downwards force by the surrounding soil.

4. Even if the soil at the lower end of a pile provides adequate supporting capacity, liquefaction of a substantial section of loose sand near the upper end of a pile foundation may permit lateral movement of the pile caps accompanied by bending or possibly buckling of the piles.

5. Vertical piles are not likely to provide any substantial degree of resistance to translation of a building due to lateral slippage in the underlying soils.

Lateral Pressures

Retaining walls should be designed to withstand seismic forces developed in the backfill materials in addition to the usual static pressures.

Utilities

Special precautions should be taken in the design and construction of utility systems to ensure that they are sufficiently flexible to withstand the ground motions induced during an earthquake. In addition precautions such as automatic shut-off valves should be nstalled to minimize hazards in the event the system s disrupted.

In the light of the above list of desirable precautions, t is clear that construction of buildings on fills over- ying bay deposits involves considerably more poten- al hazards than construction on rock or dense, hard oil deposits. On the latter materials in areas away rom the fault trace, the only hazard resulting from an arthquake is likely to be the shaking induced by the round motions. On the other hand, for earth fills over ay deposits, major hazards to be considered in ad- ition to ground shaking include subsidence and settle- ent due to compaction, landslides due to various ossible causes and liquefaction.

However, the existence of potential hazards does not ean that adequate design measures cannot be taken reduce their significance to acceptable proportions, in all types of engineering structures. In the case of ay fills at the present stage of technology, this is kely to be more difficult in the prevention of land- ides than in minimizing the possibility of liquefaction new fills or the effects of subsidence due to compaction, and special care is therefore required in evaluating landslide possibilities. Nevertheless, if suitable engineering solutions to the design problems listed above are developed, there is no reason why construction on bay fills should not be considered as satisfactory from a seismic hazard standpoint as other building sites in the bay area.

CURRENT RESEARCH AND DESIRABLE OBSERVATIONS

At the present time there is a considerable amount of research activity on soil behavior during earthquakes which is leading to a better understanding of the problems involved and better methods for evaluating and analyzing these problems. These activities include:

1. Detailed studies of soil failures in previous earthquakes leading to more valuable experience records by which to judge the potential hazards of other site conditions.
2. Analyses of ground response during earthquakes.
3. Analyses of soil liquefaction during earthquakes.
4. Studies of slope stability during earthquakes.
5. The development of new methods of testing soils to evaluate the significant characteristics determining their behavior during earthquakes.

In view of the previously limited knowledge of these subjects and the potential hazards involved, it is desirable that soil engineers responsible for the design and construction of bay fills should keep in close touch with these developments.

Since field observations play such an important role in the development of design practice, it is also desirable that detailed observation be made of the behavior of filled areas during future earthquakes. These observations should include records of ground motions in filled areas and records of corresponding motions on nearby rock outcrops. Suitable accelerometers can now be installed at a cost of about $5,000 to $6,000 per instrument, but it seems likely that within one or two years the development of new instruments will reduce these cost figures by perhaps 50 percent. Detailed analysis of such instrumental records and other aspects of the fill performance will provide a valuable guide to the probable behavior of the fill in subsequent earthquakes, possibly of larger magnitude, and thereby provide an improved basis for design and construction procedures.

SEISMIC RISK TO BUILDINGS AND STRUCTURES ON FILLED LANDS IN SAN FRANCISCO BAY

By **KARL V. STEINBRUGGE, Structural Engineer**
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SEISMIC RISK TO BUILDINGS AND STRUCTURES ON FILLED LANDS IN SAN FRANCISCO BAY

By Karl V. Steinbrugge

INTRODUCTION

In recent years, authorities from various scientific and professional disciplines have been in controversy over the seismic risk to structures located on San Francisco Bay fills. Statements have been made, or implied, that range from serious damage and destruction of all buildings and structures on bay fills to predictions of superior performance—even superior to structures founded on any other foundation material than bay mud. It is one purpose of this report to review portions of the background history, as well as to review certain aspects of the present state of the art of structural design on bay fills, and to do this in nontechnical language.

A second purpose of this report is to delineate some of the factors that determine the various types of seismic risk from the structural engineering standpoint. Seismic risk depends on many variables such as the type of construction, as well as the type of soil beneath a structure. Seismic risk is also one of viewpoint; for example, serious building damage does not necessarily require an equal hazard to life.

Detailed engineering design methods for the construction of foundations on structurally poor soils have been treated in many scientific and professional books and papers, and a discussion of them would be out of place in this report. The additional problem of seismic design of structures founded on fills over poor soils such as bay muds, is a changing problem due to the rapidly developing knowledge of soil response to earthquake forces. Professor Harry Seed in a companion report discusses the response of various types of soils to earthquake forces, while this report is oriented toward the performance of buildings on these soils.

Seismic risk, or earthquake hazard, is a major problem in the design and construction of bay fills and buildings on these fills. As a basis for evaluating seismic risk, it is first necessary to establish the probability of a damaging or disastrous earthquake occurring within reasonable limits of time. Secondly, seismic risk involves the stability of the fill in the event of an earthquake. This subject is covered in a companion report by Professor Seed, and will be given a brief, generalized treatment from the structural engineer's standpoint in this report. Thirdly, the performance of structures on bay fills will vary with the stability of the fills, materials used in building construction, the physical dimensions of the structures, and the nature of the ground motions. It is important to restate that the seismic risk to life may be quite different from that to property.

This report has made extensive use of material gathered by the author for a planned monograph on "Earthquake Hazard in the San Francisco Bay Area: A Continuing Problem in Public Policy," to be published by the Institute of Governmental Studies at the University of California, Berkeley. Seismic risk on bay fills is only one phase of the earthquake hazard in the San Francisco Bay Area.

FREQUENCY OF DAMAGING EARTHQUAKES

First, in any seismic risk determination, it is necessary to have a reasonable estimate of the frequency of damaging earthquakes. Elaborate plans are under way by the Federal government that may some day develop a reliable method for earthquake prediction. However, at the present, it is not possible to accurately predict the time, place, and size of an earthquake.

The best method at present for determining the earthquake frequency is to review past seismic history, and to judge accordingly.

One study of the seismic history of Alameda, Contra Costa, Marin, San Francisco, San Mateo, and Santa Clara Counties made for the author, showed that about 12 damaging earthquakes have occurred per century during the brief historical record. By "damaging earthquake" is meant one that caused damage in at least one location; by no means does it imply that the shock was necessarily felt throughout the entire San Francisco Bay area. The 12 earthquakes per century were not evenly spaced in time, strength, or geography. For example, the following is a list of all recent damaging earthquakes centered in the bay area and adjoining counties:

April 25, 1954: Watsonville
September 4, 1955: San Jose
October 23, 1955: Walnut Creek
March 22, 1957: San Francisco (Daly City)
September 14, 1963: Chittenden

There have been no damaging earthquakes since 1963, and no periodicity is indicated from these data.

It is more pertinent to study the occurrence of great earthquakes than the more frequent moderate shocks, since it is the great earthquake that determines the full scope of seismic risk. The historical record has five entries that are generally accepted to be the largest shocks in the bay area in historical times. These are as follows:

June 10, 1836, on the Hayward fault. At 7:30 a.m., cracks and fissures opened up along this fault from San Pablo to Mission San Jose.

June, 1838, probably on the San Andreas fault. A fissure was described and has been associated with the San Andreas fault from near San Francisco to near Santa Clara.

October 8, 1865, probably on the San Andreas fault. Considerable damage occurred in San Francisco. The earthquake presumably had its epicenter on the San Andreas fault in the Santa Cruz Mountains.

October 21, 1868, on the Hayward fault. At 7:53 a.m., cracks and fissures from this earthquake formed from about San Leandro to about Warm Springs. Damage was extensive in sections of San Francisco, and was very heavy in the town of Hayward.

April 18, 1906, on the San Andreas fault. At 5:13 a.m., the well-known San Francisco earthquake occurred. Faulting extended from southern Humboldt County to near San Juan Bautista in San Benito County.

Undoubtedly all of these earthquakes were of different Richter magnitudes, but it appears that all were of sufficient size to approach or equal the maximum probable earthquake intensities to be expected on at least major sections of San Francisco Bay area. Instrumental data do not exist for the pre-1906 shocks, and the historical data are also meager for the 1836 and 1838 shocks.

We may also note from the foregoing tabulation that the time distribution of major earthquakes is not at all uniform. This has also been noted elsewhere in the world on a larger time scale. For example, the time distribution of Japanese earthquakes since 684 A.D. has three periods of great seismic activity, with periods of lesser seismic activity between. This uneven time distribution is an important additional unknown in the estimation of earthquake frequency based on the historical record.

The foregoing tabulation shows that the largest bay area earthquakes are related to major known geologically active faults. However, data from seismic instruments show that small earthquakes in the bay area do not necessarily fall on known faults (figure 1).

In general in the western United States, large earthquakes with large surface ruptures can be related to clearly defined geologic faults. While geologists cannot guarantee that there are no major active faults under the bay except possibly for the northerly extensions of the Hayward and Calaveras faults under San Pablo Bay, it seems unreasonable to postulate a major seismic event on an unknown fault having surface rupturing under the bay.

Based on the foregoing, seismic risk is therefore essentially confined to the vibratory effects on bay sediments and man-made bay fills, and to the vibratory effects on buildings located on these soils. If geologists deem the northerly extensions of the Hayward and Calaveras faults to be potentially active, it would be imprudent to allow the development of fills in these fault zones, since stable man-made structures are not economically feasible under these conditions and since life safety normally would be impaired even with special design precautions.

It should be added that the movements on the prin cipal active faults in the bay area have been of the strike-slip type (i.e., the ground surface on one side moving horizontally with respect to the other), with very small vertical components. Similar ground move ments can be expected in the future. Thus there is no need to be concerned about an otherwise stable earth fill disappearing beneath the bay, due to a vertica drop of bedrock over thousands of square miles a occurred in Alaska in 1964 and in Chile in 1960.

Returning to the problem of earthquake frequency it has been repeatedly shown by precise triangulatio carried out by the U. S. Coast and Geodetic Survey that the earth's surface in the San Francisco Bay are is being strained by measurable amounts. For example the Farallon Islands west of the Golden Gate ar moving northwesterly with respect to Mt. Diablo i Contra Costa County. The direction of relative move ment is similar wherever measured across the Sa Andreas fault, namely that the west side is goin northerly with respect to the east side of this faul

It is a commonly held theory that when the strain become too great within the earth's crust, then ruptur will take place. A sudden rupture generates seismi waves which constitute an earthquake. This rupture normally termed faulting, will occur along the weak est zone in the earth's crust. This weakest zone normally one which has had previous movements o it, and this zone is called a fault zone. The poin beneath the earth's surface where the faulting fir starts is defined as the focus, while the point on th earth's surface directly above the focus is the earth quake's epicenter.

The measured accumulating strains in the earth crust in the San Francisco Bay area have allowe the computation of earthquake frequency estimates b dividing the average of the largest fault displacemen in the 1906 earthquake by the average rate of accumu lating strain to get the probable number of years b tween earthquakes. This number formerly came out

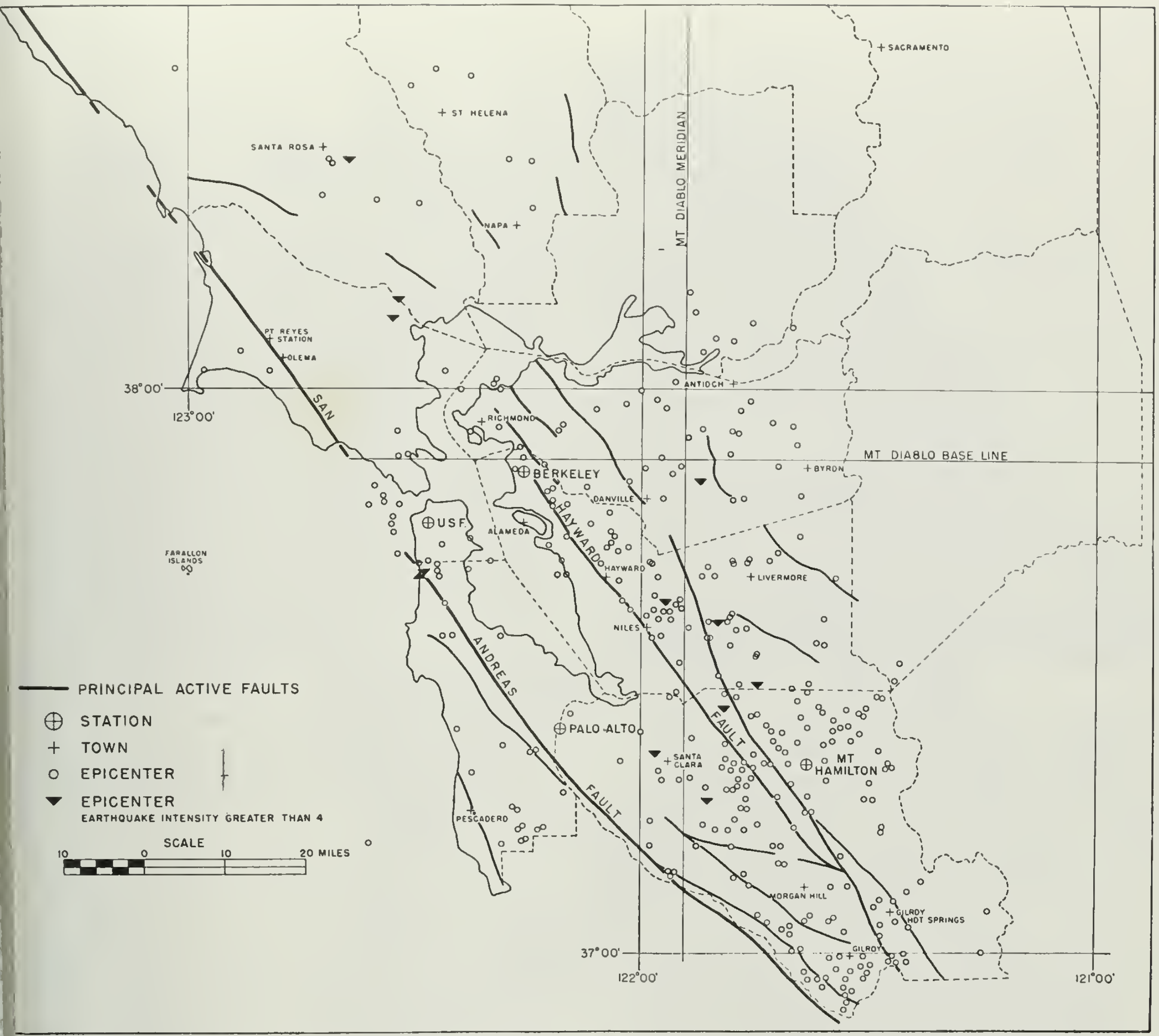


Figure 1. Epicenters of bay area earthquakes, 1930–41 and 1947–48. Source: Reproduced by permission from Perry Byerly, "History of Earthquakes in the San Francisco Bay Area," *Geologic Guidebook of the San Francisco Bay Counties* (California Division of Mines Bulletin 154, 1951).

a reasonable figure of 60 to 75 or more years, depending upon the interpretation of the data. However, with the very recent discovery of fault creep from San Pablo through Fremont on the Hayward fault, the earthquake prediction calculations are open to greater question. It is now known that the Hayward fault slipped, or faulted, for about half a foot in the City of Fremont some time between 1949 and 1957 without a recorded earthquake; no movement is now being observed. Other sections of the Hayward fault are performing differently; for example, movements are currently damaging the University of California stadium in Berkeley. Details of this fault slippage may be found in the Bulletin of the Seismological Society of America, Vol. 50, pp. 257/323 (April, 1966). We can conclude that the strains in the bay area are continuing to accumulate, although a part of them are probably being released by fault slippage. It is reasonable to assume that the large historic earthquakes and the recent fault slippage represent typical characteristics of the Hayward fault, and that despite gradual release of part of the strain, earthquakes are to be expected on this fault in the future.

The foregoing discussion is a brief overview of the problems inherent in predicting an earthquake for the San Francisco Bay area. On the basis of a thorough study of the historical record and in view of the accumulating strains, it is reasonable to expect a maximum intensity earthquake in the San Francisco Bay area once in every 60 to 100 years. This is sufficiently frequent to require that all structures and fills in the San Francisco Bay be designed to resist the forces of a great earthquake, such as those originating from the San Francisco 1906 shock.

The San Francisco 1906 shock is the largest earthquake in the bay area for which detailed information is available. It is a fair question to ask if this is the largest credible earthquake to be expected in the future. This problem has been reasonably well answered by seismologists who, by calculations made from their seismographic records, can determine an earthquake's magnitude. The magnitude of an earthquake is a measure of its size; the numerical value of the magnitude, which is on a logarithmic basis, is related to the energy released by the earthquake. The largest earthquakes in the world recorded by seismographs for over half a century are not much greater in magnitude than that of the 1906 San Francisco shock, and therefore the forces generated by the 1906 San Francisco earthquake appear to be a reasonable upper limit. The duration of the damaging intensities, however, may be substantially longer than that experienced in 1906.

In recapitulation, for planning purposes the frequency of large earthquakes developing maximum intensities in at least major sections of San Francisco Bay may be reasonably assumed at from 60 to 100 years. (These figures should *not* be used for prediction purposes.) Further, the forces released by the 1906 San Francisco shock would also be a reasonable maximum for future great shocks, but the duration may be substantially longer.

EARTHQUAKE EFFECTS TO BUILDINGS ON BAY FILLS

Effects of the Largest Earthquakes in San Francisco Bay

The basis for much of the criticism which has been directed toward the development of and the construction on bay fills is their performance in past large earthquakes. Additionally, fills have performed poorly during earthquakes elsewhere throughout the world. It is of value to critically review in detail the historical record for the bay area.

There are insufficient data on the 1836 and 1838 earthquakes to establish firm conclusions, but the 1865, 1868, and 1906 shocks have been described in varying detail. San Francisco, being the only city having significant portions of its land area on man-made fills at the time of these earthquakes, was the location of most of the damage related to man-made fills. It is appropriate to point out that in Santa Rosa in 1906, for example, intensified earthquake damage also occurred to buildings located on the deep, unconsolidated, naturally deposited alluvial soils; indeed, it is possible for naturally deposited soils to be as hazardous as the poorest man-made fill.

The available damage data for San Francisco have been well summarized in "The California Earthquake of April 18, 1906, Report of the State Earthquake Investigation Commission" (Carnegie Institution of Washington, 1908–10, two volumes and atlas, henceforth called the "Commission Report."

Regarding the October 8, 1865, earthquake which apparently was centered in the Santa Cruz Mountains, the "Commission Report" states on page 449 of Vol. I:

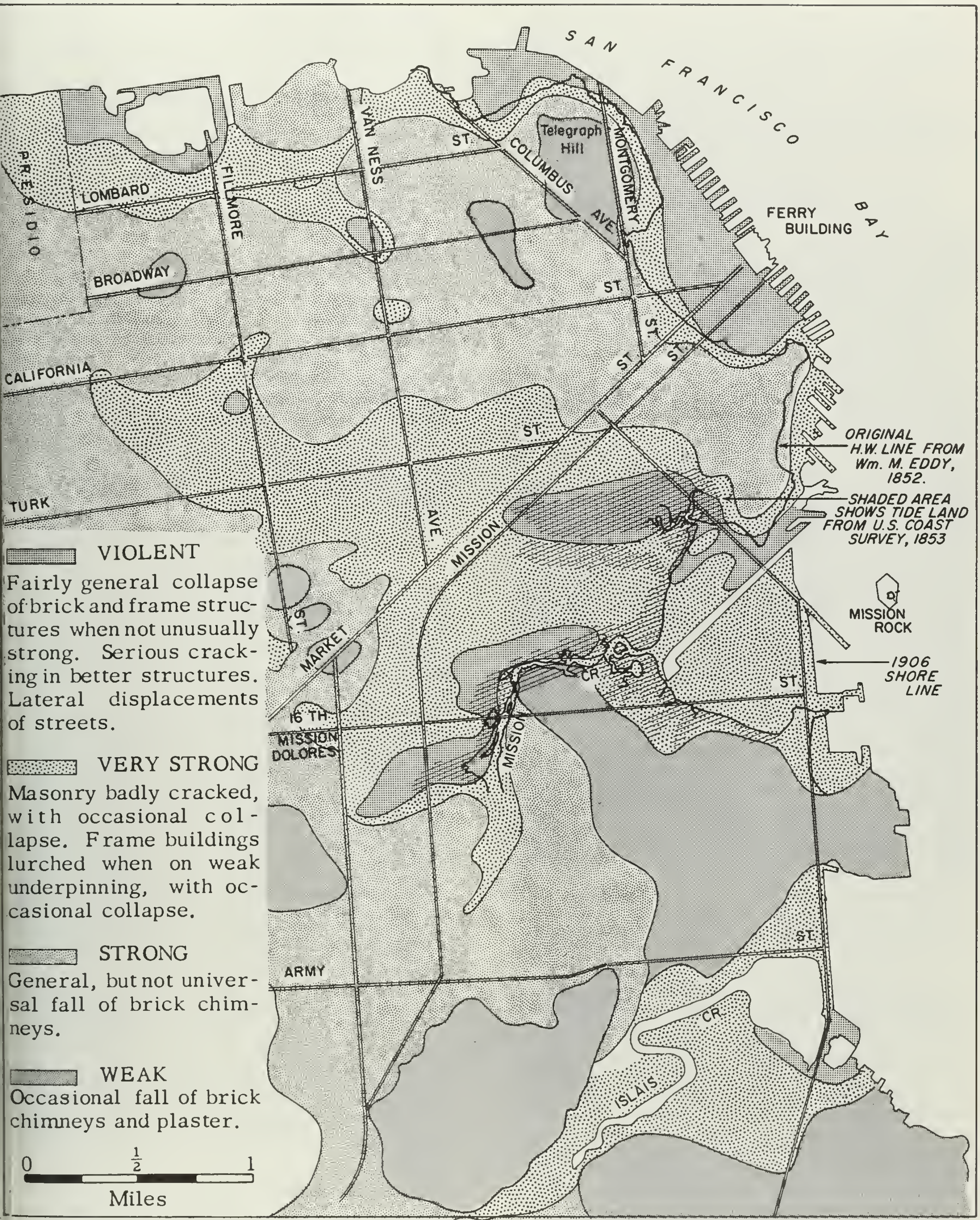
> "On the marshy lands in the vicinity of Howard and Seventh Streets the ground was heaved in some places and sank in others. Lamp posts were thrown out of perpendicular, gas pipes were broken, etc."

The October 21, 1868, earthquake on the Hayward fault also caused damage in San Francisco, and the "Commission Report" states on pages 436–37 of Vol. I:

> "The portion of the city which suffered most was that part of the business district, embracing about 200 acres, built on 'made ground'; that is, the ground made by filling in the cove of Yerba Buena. The bottom of this cove was a soft mud varying from 10 to 80 feet in depth, and the material used to fill it was largely 'dump' refuse, much of which is organic and hence perishable. Many of the buildings of that period were built flat an this filled mud, without piling, and before the land had had time to become firm. On this made land there was a very evident belt of maximum damage several hundred feet wide and running about northwest and southeast, commencing near the custom-house and ending at the Folsom Street wharf. . . .
>
> "In many places the made land settled. At the junction of Market and Front Streets, the ground sank for a foot or two, and there was evidence that the tide had risen in the adjoining lot at the same time, for a pond of water collected and remained until low tide. . . .
>
> "At the corner of First and Market Streets, the ground opened in a fissure several inches wide. At other places the ground opened and water was forced above the surface. At Fremont and Mission Streets the ground opened in many places. The general course of damage in the city was along the irregular line of the 'made land,' or low alluvial soil, where it met the hard or rocky base beneath it . . . ."

The April 18, 1906 earthquake has been studied i great detail by many authorities. Probably the mo thorough and generally accepted study of the in tensity and the relationship of damage to ground co ditions in San Francisco was made by H. O. Woo and published in the "Report of the State Earthquak Investigation Commission" (Volume 1, pp. 220/241 It is of value to quote some of the major points ma in this study:

> "About the Ferry Building, at the foot of Market Street, is a district of 'made' land shown on map (present author's note: see Figure 2 of this report), in which high intensity was manifested. Here buildings of all sorts were crowded close together. Wooden buildings, 1 story to 3 stories high, with brick or stone work fronts, were interspersed among ordinary brick buildings from 2 to 6 or 8 stories in height. Mingled with these was a considerable number of modern, class A, office buildings. Here the fire burned fiercely and caused great havoc. . . . After the fire had past, standing walls revealed ugly, sinuous cracks, in rudely parallel systems, which were not due to fire nor to dynamite. Masonry blocks in the walls of excellent modern buildings were broken as by a blow. Rivets were sheared off in parts of the framework of steel structures, and tension rods in such frames were badly stretched. Tubular cast-iron columns, supporting floor girders, were broken off near their bases in cellars where they rested upon piling. The concrete casing of piles was frequently broken. Wherever the intensity was high, the tendency to crack or crush near the base, as though a sharp blow had been struck there, was notably conspicuous. In spots the streets sank bodily, certainly as much as 2 feet, probably more. Accompanying this depression, concrete basement floors were broken and arched, as if to compensate for it. The surface of the ground was deformed into waves and small open fissures were formed, especially close to the wharves. Buildings on the water side, along East Street, generally slumped seaward, in some cases as much as 2 feet. The damage was greatest close to the water's edge, growing less as the solid land was approached, gradually at first, then more rapidly (page 233).
>
> "It is important to recognize that, despite the great intensity manifested near the water-front, first-class modern buildings, such as the Ferry Building, built on deep piling or grillage foundations, were not imperiled by injuries to their walls or framework. Some rivets were sheared off; some tension rods were stretched; an occasional girder was dis-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VAN NESS
ST
COLUMBUS AVE
Telegraph Hill
MONTGOMERY
FERRY BUILDING
BROADWAY
ST.
ST.
ST.
ST.
CALIFORNIA
ST.
TURK
AVE.
MISSION
ORIGINAL H.W. LINE FROM Wm. M. EDDY, 1852.
SHADED AREA SHOWS TIDE LAND FROM U.S. COAST SURVEY, 1853
MISSION ROCK
ST.
MARKET
CR.
1906 SHORE LINE
ST.
16 TH
MISSION DOLORES
MISSION
ST.
ARMY
CR.
ISLAIS
VIOLENT
Fairly general collapse of brick and frame structures when not unusually strong. Serious cracking in better structures. Lateral displacements of streets.
VERY STRONG
Masonry badly cracked, with occasional collapse. Frame buildings lurched when on weak underpinning, with occasional collapse.
STRONG
General, but not universal fall of brick chimneys.
WEAK
Occasional fall of brick chimneys and plaster.
0
½
1
Miles

Figure 2. Apparent earthquake intensities in different areas of San Francisco, 1906. Note large variations in intensity for areas approximately same distance from San Andreas fault which is located about 10 miles west of the Ferry Building. "Violent" areas are associated with poorest ground areas. Source: Simplified from Map 19 of the Atlas to the "Commission Report."

> lodged, and cracks were formed here and there in the brick and stone walls. Large financial loss was unquestionably occasioned, but buildings of this type were not in serious danger of collapse nor of being toppled over, either during or after the shock (page 235).
>
> "The shaking caused the materials used in filling to settle together and occupy less space, so that the surface over the whole district was lowered by amounts varying from a few inches to 3 feet or more. This is clearly seen in the change of street levels along the margin of the solid ground, where the car rails are bent downward in little monoclines. Occasionally a structure with a relatively good foundation remains at its former level, with whole neighborhood depressed about it (page 237)."

Wood concluded:

> "This investigation has clearly demonstrated that the amount of damage produced by the earthquake of April 18 in different parts of the city and county of San Francisco depended chiefly upon the geological character of the ground. Where the surface was of solid rock, the shock produced little damage; whereas upon made land great violence was manifested (page 241)."

The earthquake intensity of the 1906 shock as determined by H. O. Wood for a portion of San Francisco is shown in modified form in Figure 2. The intensity obviously varied greatly within short distances and correlates with geology.

A critical reading of the reports of the 1906 earthquake also finds statements such as these:

> "Buildings erected upon good foundations withstood the ordeal well, even when the streets around them were depressed and fissured." ("Commission Report," page 236)
>
> "Foundations did not suffer at all, no instances of damage having come to hand." ("The effects of the San Francisco Earthquake of April 18, 1906 on Engineering Constructions," in *Trans. Am. Soc. Civil Engrs.* 59:235 (1907))

The content of the paragraph including the foregoing statement indicates that the writers may have had the taller fire-resistive buildings in mind, although no qualifying statements were made.

The description of the fill materials, as well as the description of the natural marsh areas, leaves a firm impression that the made ground was obviously of poor structural quality. A most vivid description of poor ground in San Francisco was given by Hittell in "A History of the City of San Francisco" (1878) where he stated on pages 433/34:

> "The peat in the marshes that had their heads near the site of the new city hall was strong enough to sustain a small house or a loaded wagon, though a man, by swinging himself from side to side, or by jumping upon it, could give it a perceptible shiver. There were weak places in it, however, and a cow which in searching for sweet pasture undertook to jump from one hard spot to what appeared to be another, made a mistake, for it gave way under her and a gentleman hunting nearby was surprised to see her go down, and still more to observe that she did not come up again . . . Many ludricous scenes occurred in filling up the swamps. When streets were first made the weight of the sand pressed the peat down, so that the water stood where the surface was dry before. Sometimes the sand broke through, carrying down the peat under it, leaving nothing but water or thin mud near the surface. More than once a contractor had put on enough sand to raise the street to the official grade, and gave notice to the city engineer to inspect the work, but in the lapse of a day between the notice and the inspection, the sand had sunk down six or eight feet; and when at last a permanent bottom had been reached, the heavy sand had crowded under the light peat at the sides of the street and lifted it up eight or ten feet above its original level . . . so that houses . . . were carried away from their original position and tilted . . ."

The foregoing description may have been considerably exaggerated; but even so, the collapse of buildings on these soils during an earthquake would not b unexpected.

Certainly the seismic risk has been much greate to buildings and other man-made structures on ba fills. The effects of other great earthquakes throughout the world generally, but not universally, hav been similar. Experience in the bay area also show that buildings on piling perform better as a class tha do those on footings resting directly on the soil. Thes problems will be discussed by others in companio reports.

In recent years, and particularly since World Wa II, the science and profession of soils engineering ha greatly expanded its knowledge and this knowledg is being almost universally applied to major structures in the bay area. Developers of housing trac generally make extensive use of soil engineering dat Soils engineers are now able to give the architect an structural engineer vastly better data on the structural characteristics of soils and their probable earthquake performance than were available at the tim of the large bay area earthquakes discussed in th section.

Recent Experience from Moderate Earthquakes in the Bay Area

There is very little seismic experience of buildin on modern engineered fills. Only the moderate shoc of March 22, 1957, centered in Daly City which south of San Francisco can be cited. This earthquak was studied by the author and others, and a summar of their findings is as follows:

> "Cuts and fills exist throughout the Westlake Palisades tract, and some of the fills exceed 35 feet. All grading work was under the supervision of a qualified soils engineering firm. As indicated in previous paragraphs, better than 100 percent recompaction was indicated from their test samples. There was no soils engineering supervision for the minor cuts and fills made during the actual house construction. The engineering firm of Theodore V. Tronoff, Jr., evaluated the damage to each dwelling on an arbitrary scale having 10 divisions, and the weighted results of 568 dwelling investigations showed that houses on the engineered fills fared as well or better than houses on cuts (or on original ground). Further, considerably greater cuts and fills of similar soils exist to the east of Skyline Boulevard and such occasional damage as was found was not related to the soils. We may conclude that the recompacted soil was not a significant factor in the damage". . . .
>
> "In an area of ground settlement elsewhere in this tract, borings placed since the earthquake have revealed what may have been a garbage dump, covered by sand long before the housing development was started. This points out the need for numerous borings, in order to catch local problems. Also, this indicates that a detailed history of an area may be helpful in disclosing local conditions that may not be revealed by borings". . . . ("Damage to Buildings and Other Structures During the San Francisco Earthquake of March 22, 1957," by K. V. Steinbrugge, V. R. Bush, and E. G. Zacher, page 81 in *California Division of Mines Special Report #57*, 1959.)

These fills were not over bay muds or other co pressible bay soils, but certainly their performan shows that man-made fills, per se, can quite sat factorily withstand the effects of moderate ear quakes. In contrast, damage at the San Francisco

ernational Airport about 10 miles southeast of the
picenter, was attributed to both differential settle-
nents and horizontal vibratory forces. These airport
uildings were on piling.

For other recent earthquake experience of modern
lls in California, we must turn to the July 21, 1952
ern County earthquake. Highway fill settlements
ere noted on the Bakersfield-Tehachapi highway.
or example, the fill at the railroad crossing near
ehachapi settled eight inches at its western end.

During the 1964 Alaskan earthquake in Anchorage,
quefaction of certain types of soils under vibratory
rces of long duration caused spectacular land move-
ents involving many city blocks. Professor Seed,
ho is the author of a companion report, is in a posi-
on to speak authoritatively on the nature and identi-
cation of soil liquefaction. It appears that soil lique-
ction, while observed and described in the 1960
hilean and other earthquakes, was not adequately
udied until the 1964 Alaskan earthquake. This brings
the troublesome question of the seismic risk in-
lved with bay fills which have been placed in recent
ears, but without benefit of this newer knowledge
soil liquefaction. Restudying these pre-1964 fills
ay reveal unsuspected hazards, but to do so involves
e philosophically and politically troublesome ques-
on of retroactive building code laws.

STRUCTURAL ENGINEERING EVALUATION OF BAY AREA SEISMIC HISTORY

The historical records of the large earthquakes of
65, 1868 and 1906 certainly indicated, in general,
or performance of buildings on structurally poor
ils. Common exceptions were structures on piling.
mited experience in moderate earthquakes in the
y area shows that fills on firm ground can and have
rformed excellently, but data are almost completely
king on modern bay fill performance.

In view of this meager experience on modern bay
s, the laboratory and theoretical approaches must
fully utilized. These approaches are often used with
ly the judgment of the practitioner extrapolating
results to reality.

The rapid development of soils engineering tech-
ques and knowledge, including the very recent de-
opment of substantive knowledge on the poten-
lly major hazard of soil liquefaction, leaves the
iter as well as many other structural engineers with
not entirely comfortable feeling of the seismic risk
bay fills, marshlands, and other poorly-consolidated
uvial materials.

t is general practice for soils engineers to give
ound settlement figures (or differential settlement
ures), often as a function of time, to the structural
gineer for his use in the design for buildings located
poor ground. It is at this point that economics
ist be weighed against seismic risk as well as other
tors. The present state of the art does not allow a
merical equation of economics to seismic risk with-
t very substantial judgment factors. Consider two
reme and quite impractical cases:

. Remove the compressible bay soils and replace
m with firm materials (an absurd case would be replacing all soil from bedrock to the ground surface with concrete).

2. Place a structure with no special footing design directly on soils such as described by Hittel on previous pages of this report.

The first solution is impractical from a cost standpoint, while the second is impractical from a safety standpoint.

Effects of Differential Settlements on Seismic Risk

Building settlements prior to an earthquake, particularly those where one portion of a building settles more than another portion (i.e., differential settlements), can cause measurable strains in a building. These strains may result in stresses that substantially weaken a structure.

Figure 3 is an extreme case of differential settlements to a building constructed many years ago and prior to present day soils engineering techniques. These differential settlements have badly cracked the walls and otherwise weakened this building; nearby buildings have settled differentially to the point that doors had to be rehung in order to make them operable. It becomes obvious that differential settlements can (and normally do) weaken a building to the extent that it will perform poorly in an earthquake. In the writer's opinion, differential settlements prior to and during an earthquake are a major reason for greater observed earthquake damage to buildings on made land. The writer has had considerable personal experience to substantiate this observation. For example, in the 1954 Eureka (California) earthquake, differential settlements prior to this earthquake had caused damage in a brick wall of a major structure; moss was growing in the crack. The crack moved slightly during the earthquake ("worked") and paint

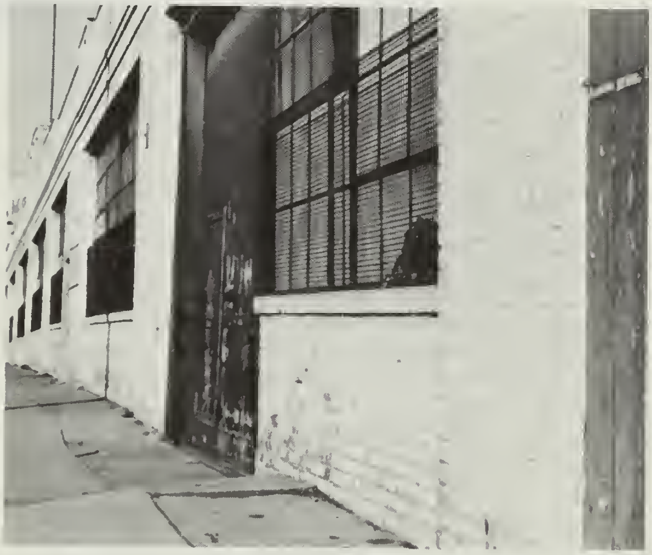

Figure 3. Effects of differential settlement on a building. Differential settlements have caused the center of the building to "hump." Note curve formed by the sidewalk and by the window sills. This building has many cracks in its brick walls due to differential settlements.

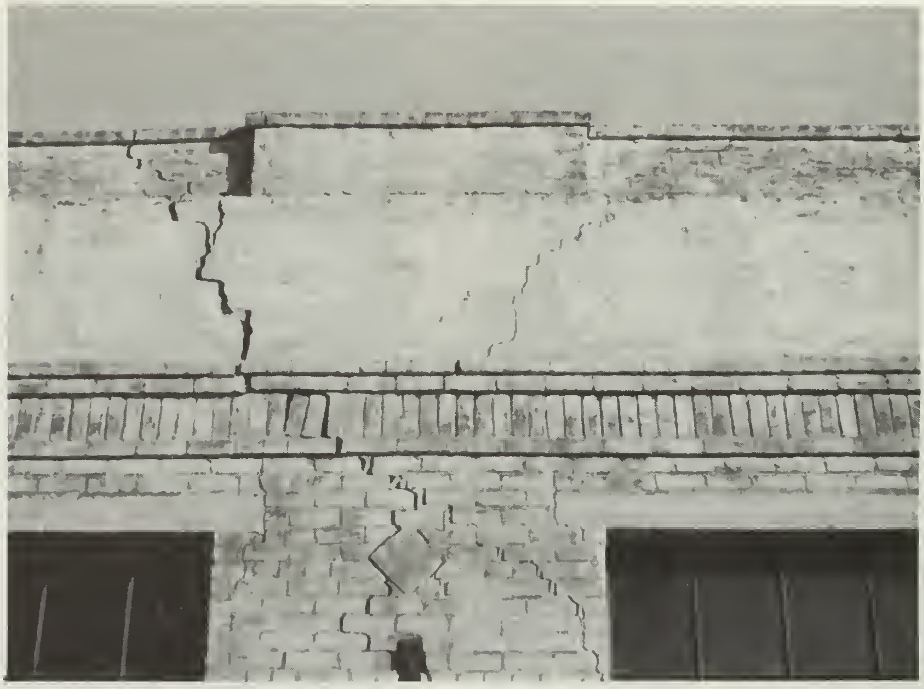

Figure 4. Settlement cracks in the wall shown in Figure 3. Cracks such as these may reduce the building's earthquake resistance.

and plaster which bridged portions of the crack fell to the floor. It was not possible to convince persons unfamiliar with earthquake engineering that the crack in the brickwork was not earthquake-induced. On many other occasions, paint deep within a crack in a masonry wall did not convince some observers that the crack had existed prior to the shock and that the flaked paint on the floor was merely the result of the failure of a subsequent coat of paint which had bridged the crack. It is not the point of this paragraph to discount the heavier damage that has been found in these structurally poor ground areas, but to explain in part the reason for its existence. The comparative lack of damage to pile-supported structures, which are not as prone to differential settlements, becomes clearer.

Even in modern structures where the theoretically predicted differential settlements perhaps actually occur as predicted, stresses within these structures are almost always increased. In a carefully made sophisticated design, these stresses can be kept within tolerable limits. If, however, actual differential settlements should prove to be greater than anticipated, then serious stresses may result. The need for experienced competent judgment which is required in evaluating anticipated differential settlements is not necessarily met by a man becoming a licensed civil engineer and opening his consulting office; the margin of judgment error in some cases can and has been considerable. Also, the structural engineer has economic pressures to increase the amount of acceptable differential settlements in his design. There is, therefore, a greater likelihood for the modern average building on bay fills to have greater pre-earthquake stresses within it than for a similar building on structurally firm so
or on rock.

The methods for minimizing differential settlemen
in buildings on bay fills vary with the type of buil
ing, as well as type of foundation. Economic conside
ations have a major role in the selection of the typ
of foundation. Piling has had overall excellent eart
quake experience throughout the world. Large mon
lithic mat footings beneath a structure have also pe
formed well; differential settlements for this found
tion type could cause a building to "tilt," affectin
elevators and other functional aspects, but not nece
sarily causing serious strains. Indeed, major buildin
in a recent Japanese earthquake tilted so far as
almost rest on their sides and yet remained structural
intact; these are, of course, extreme examples of buil
ings remaining intact after foundation failure. Ind
vidual footings which are not interconnected are t
least expensive, but are the least satisfactory.

The foregoing dealt with footing types rather th
soils beneath the footings. Today's general practi
for one and two-story buildings is to place a blank
of carefully selected and compacted soil of many fe
in thickness over the compressible bay soils; t
blanket will act as a mat and light structures wi
conventional footings resting on this mat should n
settle differentially.

It is usually the province of the structural engine
architect, and/or developer to weigh the soils eng
neer's recommendations against the estimated costs f
reducing differential settlements to some tolera
limit. This tolerable limit is a matter of judgme
and no commonly accepted standard exists. In t
many instances costs have probably influenced jud

nents, and this may explain some of the observed ifferential settlements in relatively recently placed ills.

The historical record of San Francisco Bay area arthquakes indicates that tall buildings in poor round areas have performed well. This has been only omparative; table 1 is an indication of differential ettlements which have been recorded in downtown an Francisco in poor ground areas, and suggests that gnificant stresses may have developed in these tall uildings since construction. Presumably, modern uildings will have less differential settlements because f more knowledge and better foundation design.

Ground Motions on Bay Fills

Research, particularly in Japan, is showing that the round motions are different on structurally poor rounds than on rock. The reasons for these different iotions are described in the companion papers in iis volume. The soils beneath a particular building e usually very complex, with varying soil strata aving different dynamic characteristics; practical exerience shows that bay fill soils are very complicated. general, the frequency of earthquake vibrations ow down—in other words, the ground oscillations ecome slower—as seismic waves progress from bedock through less competent materials to the building undation.

Evidence from small earthquakes also indicates that e amplitude of the ground motions becomes subantially greater on structurally poor grounds than rock; in other words, ground motions are also nplified as they pass from bedrock through less comtent materials. This amplification has been often emonstrated in a crude fashion by vibrating a bowl jellylike material.

Table 1. Total observed settlement of multistory buildings in financial area of San Francisco.

Height of building	Period	Total settlement (at various columns)	
		Maximum	Minimum
1 stories	1925–47	3″	0″
2 stories	1922–50	7″	5″
2 stories	1922–50	10″	6″

igures are to nearest inch. Data based, in part, on "Building oundations in San Francisco," by Charles H. Lee (*Proc. Am. oc. C. E.*, Vol. 79, Separate No. 325, 1953)

The foregoing factors have not yet been incorpored into American building codes, partly because e state of the knowledge is too new and too incomete. These factors theoretically can become troublesne to long period structures. In lay terms, long riod structures are most commonly the high rise ildings.

A classic example of selective damage to tall buildigs (i.e., long period structures) compared to rigid e and two-story buildings, occurred in the July 28, 57, Mexican earthquake. The earthquake epicenter s reported from 170 to 220 miles from Mexico City. Damage was pronounced to many of the tall reinforced concrete and steel frame buildings in downtown Mexico City, and several collapsed. Small one and two-story "collapse hazard" buildings performed well. Soils in downtown Mexico City consist of unconsolidated lake bed and other materials, and they are of exceptionally poor structural quality. The nature of the ground motions from this distant earthquake was such as to selectively damage the long period structures (multistory buildings), but multistory building damage was far more pronounced in the structurally poor ground areas than in the nearby firmer ground areas. Differential settlements obviously played a role in some of the damage, but the author's observations indicate that it was a minor role in most cases.

In a general sense, the seismic risk to buildings on bay fills from ground motions described in this section is least for one story light mass structures of no more than moderate floor area.

LIFE SAFETY VERSUS PROPERTY DAMAGE

Seismic risk can be viewed from different standpoints with quite different results. Severe earthquake damage to property does not necessarily incur an equal life hazard. A building can become so seriously cracked that repairs may be uneconomical; however, if building collapse does not occur, then the building's occupants may be able to leave without harm. On the other hand, the failure of small value items, such as the collapse of light fixtures and the collapse of non-reinforced hollow concrete block partitions around stairs, has caused injury and even death.

A commonly accepted viewpoint on seismic risk is to take such engineering design steps as may be necessary to reduce the life hazard to an absolute minimum, and to restrict property damage to reasonable limits in the event of a great earthquake, such as a repetition of the 1906 shock. This philosophy is embodied in most of today's building codes. There is, however, no consensus on what constitutes a "reasonable limit" to earthquake property damage. Some engineers may be willing to allow total loss to their client's property, provided that life safety is assured. This viewpoint is also sometimes held by speculative developers who, through sale of the developed property, transfer the hazard to the unsuspecting buyer.

Theoretical calculations as well as experience have shown that the seismic risk to life is usually minimum in one story wood frame dwelling structures. Total collapse very rarely occurs even when these dwellings are tossed about like boats on an open sea. Classic examples of the inherent strength of conventional wood frame dwellings could be found in the Turnagain Heights section of Anchorage after the 1964 earthquake; wood frame dwellings formerly on level ground were found at 30 degrees and greater slopes and were often salvageable. This is not to say that structures of other materials cannot and have not performed equally well, but an equal degree of safety requires considerable additional cost and engineering know-how.

Another aspect of life safety is egress from a heavily shaken structure. Panic during and following an earthquake can become a serious hazard in a multistory structure housing many people. When damage occurs to multistory structures, it commonly occurs to the stairs, to walls around the stairs, and to elevators and their walls. Debris falling on the stairs is a significant life hazard. The degree of seismic risk from panic conditions will increase as a function of the number of stories and the occupant load.

One story industrial and commercial structures often have concrete or masonry exterior walls and wood roofs. In order to achieve a minimum seismic risk to life, these types of structures must be exceptionally well tied together when located on bay fills. Unfortunately, American building codes do not require special earthquake design factors for buildings on poor soils.

In summary, the increased seismic risk to life in structures located on bay fills can be reduced by the use of light mass structures such as single family one story wood frame dwellings. The use of specially interconnected footings can also minimize the life and property risk. Industrial and commercial structures have least life risk when they are of one story and of judiciously selected structural designs and materials. Requirements to control property damage are more severe than those for life safety. It is desirable to keep the population density to low figures, and considerable thought should be given to the increased risk before allowing the construction of high rise structures, auditoriums, and similar structures in poorest ground areas. It should also be mentioned that bay fills are not the only geologic hazard to construction; landslide areas and fault zones are additional major hazards.

FIRE FOLLOWING EARTHQUAKE

Fire following an earthquake is a distinct hazard. However, fire is a substantially greater seismic risk to property than to life. Water systems necessary to fight fires have often been crippled or have failed in earthquakes, particularly in the poor ground areas described in the previous paragraphs. The reasons for this intensified water system damage are largely traceable to differential settlements and to ground lurching in a horizontal direction. Additionally, earthquake induced water surges in pipelines have been the probable cause of leaks where the lines have been significantly weakened by corrosion and other pre-earthquake causes. Water and gas pipelines entering buildings have often been broken due to earthquake induced building settlements, or due to a building rocking back and forth on soft compressible soils during the earthquake. The foregoing problems can be readily solved at virtually no cost by special attention to design details.

All power, water and gas systems entering bay fills should be designed so they can be isolated within sections of the district, and within the entire district, in an earthquake emergency. This calls for coordinated planning between the fire department and water and gas utilities prior to construction. This planning should be followed by the development of an earthquake operational procedure checked out at least onc a year by the fire department. This latter aspect ca be easily forgotten with time, and thereby negate good fire protection system.

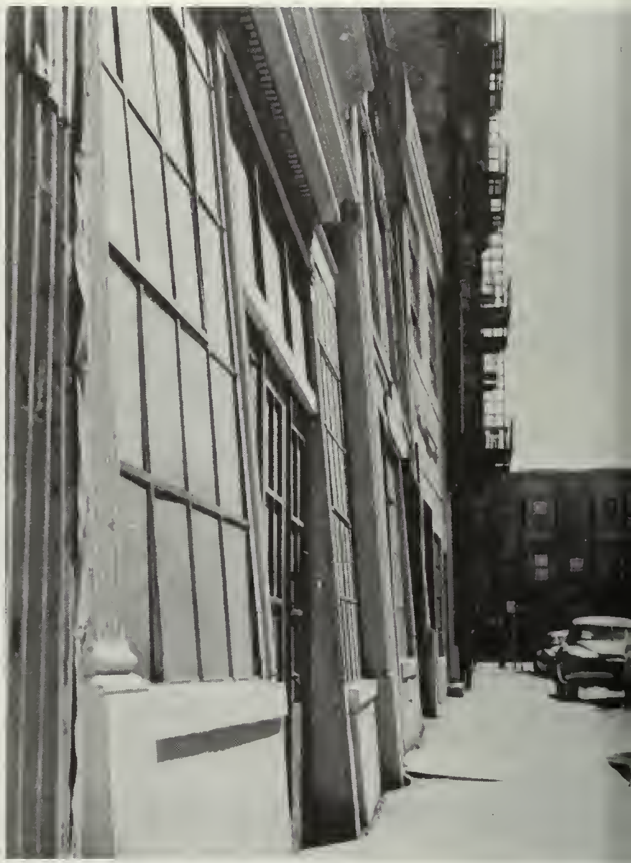

Figure 5. Building tilted due to differential settlement. Doors ar vertical, but buildings are not.

A second line of defense for large development or for fire hazardous industrial and commercial developments, or for complex piping and valving system is a special and independent water system solely fo fire protection use and which makes use of wate pumped from the bay. The City of San Francisco ha such a system.

Fuel oil and other flammable liquids stored in larg diameter tanks have suffered damage, and extensiv fires have sometimes resulted during large earthquake However, the spectacular and destructive oil fires Seward, Valdez, Whittier and Crescent City in th 1964 Alaskan earthquake were principally the resu of the seismic sea wave which destroyed the tank and distributed the oil over large areas. Significantl damage to oil storage tanks at a number of locatio in Anchorage did not result in fire; the seismic se wave did not affect Anchorage. The 1952 Ker County earthquake, with its thousands of oil tanks i the heavily shaken area, had no oil fires and ver little oil loss. The Kern County tanks, of course, we subjected only to ground vibrations and not to forc from a seismic sea wave.

Oil loss can come from tank shell failure as a resu of vibratory ground forces, or from pipe breaks

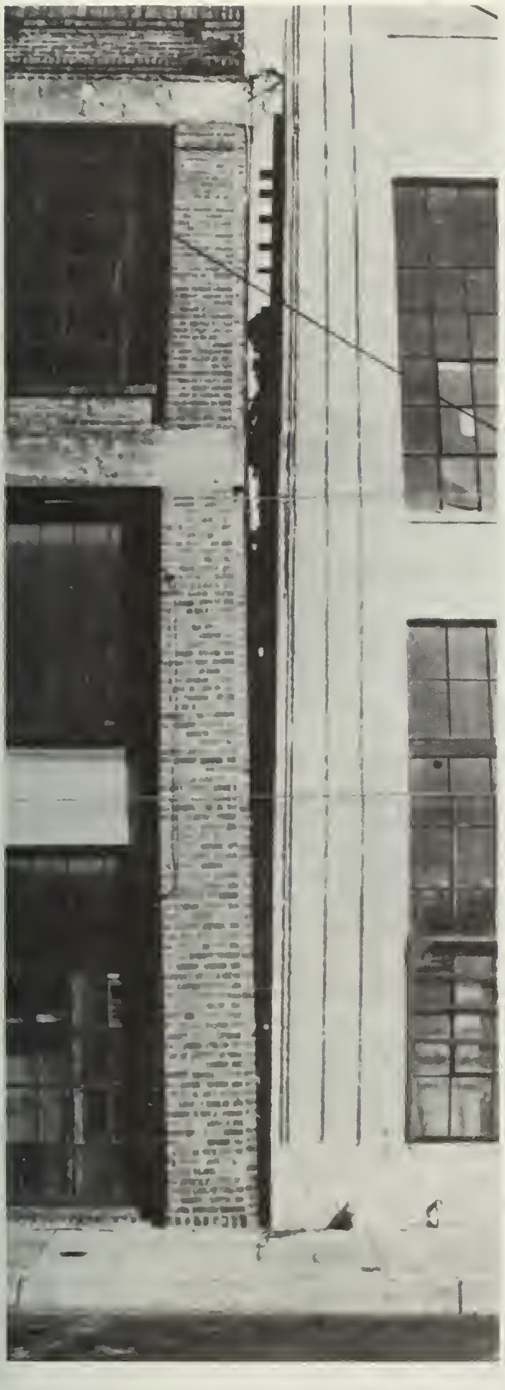

Figure 6. Separation between buildings was due to differential settlement. Location is Mission swamp area of San Francisco.

ping entering tanks, or from failure of the support-
ig bay fills, or from a combination of these factors.
he first two factors can be overcome by reasonably
vll established design methods and without signifi-
ct costs, and the customary diking system will act
a a second line of defense. The problem becomes
aite on bay fills if the fill should fail. The dikes also
a likely to fail, as an accompaniment of any major
sl failure beneath the tanks. Fire would not neces-
sily follow if no source of ignition were present.

While there is an increased earthquake risk from flammable storage located in tanks on bay fills, a judiciously located tank farm well isolated from other developments and in an area where any escaping flammable liquids would not be carried far by tidal currents or by prevailing winds could be developed without increasing the fire hazard to adjoining areas. Further minimization of the risk could be obtained if the tanks were placed on a firm foundation, preferably piling. It would seem that only a small portion of the bay could be suitable for this type of development.

POLICING

It has been mentioned in previous paragraphs that the science and profession of soils engineering for earthquake forces has come of age in recent years, but major developments are still to be expected. Their leading practitioners in the San Francisco Bay area are among the world leaders, and their judgment and experience are of the highest order.

The number of local firms in the field of soils engineering has substantially increased in recent years, and is still increasing. It is conceivable for a land developer to shop for his experts and to find one who will approve or recommend procedures involving judgment-evaluated risk that the large majority of the other practitioners would question. Admittedly, all professions require a substantial amount of judgment, and differences of opinion are to be expected among competent professionals.

Life safety in buildings is subject to a set of checks and balances in the form of minimum standards found in building codes which are enforced by local building officials. This set of checks and balances is effective to one degree or another, and includes the concrete foundations to the roof. However, the usual building department is not staffed to review the soil engineer's report. Further, the state of the art of soil response to seismic forces appears to be such that no generally accepted set of minimum standards exists. Quite possibly the best set of criteria of what should be expected in a soils engineer's report may be found in "San Francisco Bay Area Rapid Transit District, Soil Investigations," Report of Interim Soil Mechanics Advisory Committee to Parsons Brinkerhoff—Tudor-Bechtel, July 1, 1963.

It seems, therefore, that the higher risk to buildings on bay fills receives less "check and balance" attention than do the superstructures. It is a fair question to ask if this area should not receive more supervision from the building departments.

SUMMARY AND CONCLUSIONS

Earthquakes strong enough to cause damage have occurred about 12 times per century in the San Francisco Bay area. The past history of these earthquakes shows no predictable pattern as to time, strength or location. Large earthquakes have occurred five times in the Bay area since 1836, the latest in 1906. Measured accumulating strains in the earth's crust indicate the inevitablility of future earthquakes in this area. At present, earthquakes cannot be predicted as to time, place, or severity. However, based on all available evidence, it is reasonable to expect a large earthquake in the San Francisco Bay area once in every 60 to 100 years. These figures should be used as a basis for planning, and not for predicting. This frequency is sufficient to require that all structures and bay fills be designed to resist the forces of a great earthquake.

The historical record of large earthquakes in the San Francisco Bay area certainly shows a greater seismic risk on bay fills than on firmer soils or on rock. However, a reading of this record clearly indicates the structurally incompetent nature of the soils involved in the examples of accentuated damage on bay fills. It also points out that some buildings survived excellently, particularly those on piling. The historic record is vitally important, but it must not be extrapolated to different soil conditions without careful engineering analysis and judgment.

Fills constructed by present-day methods on compressible bay soils (including "bay mud") are quite different from those which are discussed in the historic earthquake record. Experience with the performance of these modern fills under seismic conditions is very small. However, this experience plus experience from non-bay fills certainly shows that modern well compacted engineering fills perform much better than did the uncontrolled fills of recorded earthquake history.

Our knowledge of the seismic behavior of soils is rapidly increasing, but judgment still must play a very important part. The fact that rapid progress is being made in the understanding of these unknowns, however, leaves many structural and civil engineers with not as comfortable a feeling as might be wished since future developments may raise questions regarding the safety of present construction. Unfortunately, a conservative approach usually involves greater costs.

Settlement of bay fills, especially differential settlements, can seriously weaken a building prior to a shock. This allows earthquake forces to more readily damage the weakened structure. The control of differential settlements in a bay fill is a function of economics in:

a. The construction of the fill, and

b. The construction of buildings on the fill.

Steps to reduce differential settlements, such as the replacement of the poorest soils by engineered fills, surcharge loadings for long time periods to accelerate compaction and settlement prior to building construction, as well as other methods can significantly raise the cost of land development. There is a natural tendency for a land developer to save money and, possibly in some cases, to select a soils consultant who will give him the cheapest solution.

The design of a building on filled land also involves economic factors of seismic risk because the building must have special features to accommodate the anticipated differential settlements. This may be done by piling, large mat footings, heavily interconnected spread footings, special beam to column connections, and the like. The foundation designs for major structures are often quite sophisticated as well as expensive. Economic pressures may exist to allow some degre of differential settlements within a structure at th possible cost of decreased safety factors.

Building codes are intended to establish minimu (not maximum) standards of safety. A building th just meets all of the minimum standards of a buildin code is, in effect, on the verge of being legally unsaf Policing for compliance with building code requir ments is the province of the local building depar ments. However, it is not customary for buildin departments to have personnel qualified in soils eng neering on their staffs. Therefore, for any specif project, the conclusions and recommendations of th soils engineer rarely receive the same scrutiny as d the drawings for constructing the building. With th large increase in the number of soils engineering co sultants, the superior performance of past years ma not always be maintained in all cases. It seems reason able, therefore, to critically review present practic to see if it is appropriate to recommend some i creased degree of policing of bay fills.

Meanwhile, a conservative approach is in order t compensate for a different seismic risk on bay fill Uses of filled land over deeper compressible soi should be restricted to minimize population densit Multistory structures with many occupants when pe mitted, must require special design attention to th foundations as well as to the superstructure by exper enced licensed structural engineers. Water and g systems should be specially valved in bay fill areas t minimize the effects of pipe breakage. An independe water system for fire protection using bay water desirable for particularly hazardous occupancies ar locations.

From the earthquake standpoint, the use of eng neered fills over deep bay compressible soils for buil ing foundations must be considered to be experiment to some degree. Many answers regarding the degr of increased seismic risks and methods to reduce it c be derived from the records of seismic instrume after even moderate shocks. In the interests of lo range land use and planning, strong motion seism equipment should be installed and housed on ea major bay fill development as well as on other kin of natural and artificial ground. Each installati should be coordinated with and become a part of t U. S. Coast and Geodetic Survey network for b results. One proposal is to install and house these i struments at the expense of the developer, with t condition that the records remain public. Somewh similar programs for buildings are already in existe in California, and the City of Los Angeles can cited as the principal example.

Implementing the foregoing recommendations quires the establishment of a number of value ju ments in an area where there is little or no precede As a starting point, it is recommended that all pot tial developable San Francisco Bay fill areas be zon into four categories, and that a map showing th zones be available for distribution to the public. broad outline of some of the special requirements r ommended for these zones are:

one 1: Minimum increased risk zone:

All structures three or more stories in height shall have a structural engineer of record who, upon completion of the construction and before occupancy, will file a statement with the local building department that he or a structural engineer on his staff, has knowledge that the structural engineering aspects of the plans and specifications have been complied with during construction. (Precedence for this exists in California public school construction. An undivided responsibility for a building's structural integrity will lead to better construction and compensate for the increased risk.)

one 2: Moderate increased risk zone:

All requirements of Zone 1, plus the following:

Single family and multiple family occupancies shall be limited to two story wood frame construction. Other occupancies shall be no higher than one story plus mezzanine.

A structural engineer of record is required for all buildings having concrete, brick, and/or hollow concrete block walls in which these walls constitute more than 25 percent of the total exterior wall area. No city halls, hospitals, or places of detention are allowed in this zone.

one 3: Substantially increased risk zone:

All requirements of Zone 2, plus the following:

Occupancies shall be limited to warehouses, storage areas, and similar minimum population density occupancies. Offices shall be only those necessary to facilitate these occupancies. Dwellings are not permitted.

Structures shall be one story in height.

Only public buildings which are necessary to service the area are allowed. Fire stations could be placed in this zone if mandatory from other considerations.

Zone 4: Maximum increased risk zone:

Parks, golf courses, and similar land use is permitted. Appurtenant structures (such as storage sheds) are permitted if of one story wood frame construction. (This zone could include former refuse dump areas.)

Zone 1 may constitute most of the developable land, depending on how much land is set aside and the engineering qualities of the land. Special provisions for waivers must be incorporated into any rules; obviously, even in Zone 4 it would be possible (but not necessarily economical) to put in special foundations capable of overcoming site deficiencies.

Technical and detailed specifications involving several disciplines will have to be established in order to equitably zone the bay and to further implement the foregoing. Guidelines for differential settlements may be established if feasible. Existing soil data will have to be evaluated for each zone. Criteria for the locations of seismic instruments are required. Minimum standards for soil reports and the policing of engineered fills will have to be considered. Detailed standards for special risks such as oil storage tanks will have to be developed.

The scientific and technical implementation aspects of the foregoing should be the work of a professional group and not that of an individual since judgment plays such an important part.

THE SAFETY OF FILLS

By BOARD OF CONSULTANTS

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THE SAFETY OF FILLS *

By Board of Consultants, Bay Conservation and Development Commission

INTRODUCTION

On February 16, 1968, the San Francisco Bay Conservation and Development Commission (BCDC) appointed a *Board of Consultants to Review Safety of Proposed Fills*. As a first step, this Board of Consultants began the study of the criteria which would be necessary to establish a satisfactory level of safety in a field where no generally accepted standards currently exist.

It must be expected that the San Francisco area will be subjected to damaging earthquakes during the useful life of all major developments. Since the time and severity of future earthquakes cannot be predicted with any degree of precision, it is recommended that the design of all major facilities give appropriate consideration to the occurrence of a major earthquake.

Considering San Francisco Bay as a whole, the kinds of earthquake phenomena that can cause damage are listed below, generally according to degree of hazard:

1. Shaking of the ground. This is placed first because, although to varying degree, all bay fill areas will be subjected to amplification of ground motion.
2. Ground failure. Compaction, liquefaction, lurching, slides, etc.
3. Inundation. Seiches and tsunamis.
4. Surface faulting. Probably a significant potential hazard at only a few localities.

Actually, the order could vary widely from place to place, and would also be influenced by intended use of the site in question.

It must be recognized that it is not economically or practically feasible to design all structures to preclude the occurrence of damage in the central region of major earthquakes. The intent of the design, therefore, should be to provide for a major earthquake in such a fashion that structures would not be expected to collapse and endanger the life and safety of people —even though fairly substantial property damage might occur. In some instances, it may be advisable for the designers to indicate the provisions which they believe appropriate for a more normal design earthquake and to indicate the extent of damage which they would expect to be acceptable for this type of earthquake.

The present state of the art of engineering bay fills and structures thereon requires, in many cases, the best scientific and professional judgment of geologists, soils engineers, structural engineers, and architects in order to assure reasonable standards of safety. This need for the highest engineering and scientific competence can not be overemphasized, and there is no public assurance that it exists for every development.

At present, local governments do not take an active role in these special technical problems, largely because of economic reasons relating to adequate staffing. It does not appear that this situation will substantially change in the near future.

Therefore, in any plan for the conservation and development of the bay, the BCDC Board of Consultants feels that reasonable and adequate safety criteria must be developed and be enforceable. This report is the product of the safety criteria studies made by the BCDC Board of Consultants. It is hoped that the report will provide an initial set of criteria for a successor board, which could begin with this document and refine it further as new information becomes available to it.

This report deals with the safety of fills in the area of San Francisco Bay as defined in the McAteer-Petris Act, 1965, that established the San Francisco Bay Conservation and Development Commission. The Act defines San Francisco Bay as the shoals outside the Golden Gate, the water areas of San Francisco, San Pablo and Suisun Bays, including marshlands, tidelands and submerged lands as far east as the Sacramento River line, but excluding all lands that are not subject to tidal action. Although geologic situations in other parts of San Francisco Bay area and in parts of other major urban areas of the West Coast pose problems of building safe structures, it is not the purpose of this report to compare problems of safe bay fills with problems of safe construction in other areas.

In developing criteria to control the development of San Francisco Bay front and reclaimed tideland areas, it is recommended that the following reasoning be accepted as a guide for the future safety of such developments, including the effects of earthquakes.

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* Ed. Note: "The Safety of Fills" was submitted while the rest of this volume was in press.

The construction of buildings on fills overlying bay deposits or comparably weak natural soil deposits involves a greater number of potential hazards than construction on rock or dense hard soil deposits. On the latter type of materials in areas away from a fault trace, the only hazard resulting from an earthquake is likely to be the shaking induced by the ground motions. On the other hand, for earth fills over bay deposits or other natural weak soils, major hazards to be considered in addition to ground shaking include subsidence and settlement due to consolidation or compaction, landslides due to various possible causes, and liquefaction. Hazards due to displacement along a fault or due to inundating as a result of seiches or tsunamis exist regardless of material. However, the existence of potential hazards does not mean that adequate design measures cannot be taken to reduce their significance to acceptable proportions. In the case of bay fills at the present state of technology, successful application of such measures is likely to be more difficult in the prevention of landslides than in minimizing the possibility of liquefaction in new fills, or of the effects of subsidence. Special care, therefore, is required in evaluating landslide possibilities.

There is a wide range of hazards and risks to life and property on bay fills. These hazards vary with the different geologic and foundation conditions, the use of the areas to be created, and the type of structures to be constructed on these areas. The recommended approach is to classify these risks by zones and occupancies. The permitted occupancies, the required geologic and engineering invesigations, and the type of construction for each zone is then established.

Geological and seismological considerations related to construction on bay fills are described in the first part of this report. The details of the required geologic and soils investigations are set forth in the second part. The third part relates the type of such investigations with the risk zones and the desirable occupancies.

The structural system employed for any proposed building should be designed in accordance with established principles of mechanics and structural design. It should survive the test of rational analysis, taking into consideration all forces, both vertical and horizontal, including both uniform and differential settlement, and building and ground motion response.

GEOLOGICAL AND SEISMOLOGICAL CONSIDERATIONS

Record of Effects of Historic Earthquakes

It is clear from the record of historic earthquakes that the potential for damaging effects to man-made structures is greatest in areas underlain by wet, geologically young, unconsolidated sediments (San Francisco, 1906; Tokyo, 1923; Long Beach, 1933; Fukui, 1948; Arvin-Tehachapi, 1952; Chile, 1960; Alaska, 1964; and Niigata, 1964).

Phenomena associated with historic earthquakes that seem particularly pertinent to development of criteria for acceptably safe bay fill developments are discussed below.

Stability of Embankments

Generally, the record of the stability of embankments on wet, water-laid, geologically young unconsolidated sediments is not good. In past earthquakes, it has been common for embankments to fail by settling (as much as several feet) into the underlying sediments and by ground cracking. Cracks occur not only in the embankment more or less parallel with its longitudinal axis, but also in a zone as much as several tens of feet wide, adjacent to the embankment. Following are a few examples of failure:

1906 Earthquake

J. H. Wallace, H. C. Phillips, R. M. Drake, and E. M. Boggs in the "Report of Committee on the effect of the earthquake on railway embankments", American Society of Civil Engineers, Proc. Vol. 33, state:

> "Embankments across marshes, or with soft strata underlying them, settled more or less. In some cases the settlement was vertical; in other cases there was considerable horizontal with the vertical movement.
>
> "At one point on the marsh between Benicia and Suisun on the Southern Pacific, the settlement was 11 ft.; at another point, 5 ft. These were nearly vertical.
>
> "Between Niles and San Jose, on the Southern Pacific, there was at one point a displacement of 3 ft. horizontal, but the vertical displacement was only 6 in.
>
> "On the North Shore, about 2 miles north of Point Reyes, the road, originally, had been constructed with pile trestles across several arms of Tomales Bay; these trestles had been filled about 15 years ago, the roadbed being about 8 ft. above ordinary high tide. Two of these embankments, 2,200 ft. and 900 ft. long, respectively, sank until the water at high tide washed over the rails.
>
> "Settlements of this character, and of more or less severity, were common on all railroads operating about the Bay of San Francisco and its tributaries."

Fukui Earthquake, 1948

In "Vol. II, Engineering" of the report, "The Fukui Earthquake, Hokuriku Region, Japan, June 28, 1948," it is stated that "Damage to railroad embankments was substantial where they crossed soft ground. They suffered subsidence, upheaval and fissuring."

Alaska Earthquake, 1964

McCulloch and Bonilla state in their report, "Railroad Damage in the Alaska Earthquake," in the September 1967 issue of the Journal of the Soil Mechanics and Foundations Division Proceedings of ASCE:

> "Damage was also caused by a loss in the bearing strength of unconsolidated sediments, which allowed vertical settling of the railroad embankment. In some areas the whole embankment settled irregularly from about 2 ft. to 6 ft. into the underlying sediments. Comparison of design profiles with postquake profiles across the embankment suggests that there was minor spreading of the embankment fill, but that most of the lowering was due to a downward movement into the underlying sediment."

Lurching, Etc.

Lurching, slumping, landspreading toward a free face and associated cracks are characteristic phenomena in water-saturated sediments in all earthquakes arge enough to induce significant ground motion (San Francisco, 1906, M = 8¼; Hebgen Lake, Mont., 1959, M = 7.1; Alaska, 1964, M = 8.4). As well as opening of cracks in the ground, vertical displacement along cracks as much as a few feet may also occur. Following are two examples of these phemonena associated with the 1906 earthquake. Both localities are underlain by fine-grained sediments.

Coyote Creek

On page 281 in the report of the California Earthquake Commission, it is stated:

> "At the bridge over Coyote Creek, on the Alviso-Milpitas Road, the concrete abutments were thrust inward toward each other about 3 feet. A pile driven in the middle of the stream, which had been cut off below the water level, was lifted about 2 feet and now rises above the water.
>
> "About 150 feet north of this bridge the banks of the stream cracked, the fissures running parallel with the channel and the land on the creek side sliding toward the stream . . . .
>
> "In the road running northward along the west side of Coyote Creek from the bridge, many large cracks opened. Five hundred feet north of the bridge the cracks were 2.5 feet wide and 3 feet deep when the place was visited April 26. Farther north the cracks were very abundant, mostly parallel with the road, and some were 4 feet deep and 3 feet wide. A quarter of a mile north of the bridge, the whole road was shoved eastward into the channel of the creek, and with it a large number of willows and cottonwood trees that grew along the banks. Just south of this place the road was broken up badly for a distance of 300 feet. One of the largest cracks in the road was 5 feet wide, 6 feet deep, and more than 100 feet in length . . . . For the most part the principal features were approximately parallel with Coyote Creek."

Tomales Bay

Similar shifting of sediments took place in Tomales Bay. On page 79 of the report of the California Earthquake Commission, it is stated:

> "There was also a horizontal shifting of mud over a considerable area. Residents familiar with depths of water in the vicinity of Inverness stated that the earthquake caused a decided shoaling along the coast, but that the relation of water levels to firm ground was unchanged. It was also stated that a channel which had existed parallel to the west shore of the bay, and to which piers had been run, was obliterated by the earthquake. The shoaling might have been caused either by an uplift of the bottom or by shifting of the mud of which it is composed toward the shore."

ubmarine Slides

The Alaska earthquake triggered numerous subqueous slides along the outer edges of thick prisms f water-saturated poorly consolidated sediments in ords and in some lakes. Serious damage and loss of fe resulted not only from the slides but also from ne water waves generated by the slides at localities ich as Seward, Portage, and Valdez.

quefaction

Liquefaction in sands and other fine sediments (San rancisco 1957, Anchorage 1964, Niigata 1964) can be a very important factor in causing damage from foundation failure and landslides.

Seismic Sea Waves and Seiches

Seismic sea waves may occur at great distances from a major earthquake (Crescent City, California, San Francisco Bay—Alaska earthquake 1964; Half Moon Bay, California—Chile earthquake 1960).

Most of the damage in San Francisco Bay from the tsunami generated by the Alaska earthquake was to small boats and berthing facilities in Loch Lomond Harbor in San Rafael, according to Spaeth and Beikman in Coast & Geodetic Survey Technical Report No. 33. According to tide gauge records, the tsunami arrived during falling tide. At the Presidio, the maximum rise was about 3½ feet above normal tide level and the maximum fall, which occurred about an hour earlier, was 5 feet below normal tide level. Corresponding rise and fall figures at Alameda were about 2½ feet and about 3.7 feet.

The extreme case of a tsunami reported generated by a California earthquake is that reported associated with the December 21, 1812 earthquake. According to Senora Juana Briones in the report, "Descriptive Catalog of Earthquakes of the Pacific Coast of the United States, 1769 to 1928," by Sidney D. Townley and Maxwell W. Allen, Seis. Soc. Amer. Bull., v. 29, No. 1, January 1939, ". . . the earthquakes were so severe as to cause tidal waves which covered the ground where the plaza now is [San Francisco]."

The matter has been recognized as of great importance and has been the subject of a detailed study, "A Review of the Evidence for the Santa Barbara Coast Tsunami of December 1812," by *Marine Advisors* (V. J. Grauzinis, July 1965). The conclusion was, "The evidence seems to point to a nominal tsunami, not generally—if at all—witnessed among coast residents."

According to Townley and Allen, rising and falling of water in San Francisco Bay is reported to have accompanied two earthquakes in 1854 and one in 1856.

Tectonic Displacement

Regional tectonic uplift and subsidence of several to a few tens of feet have been associated with historic earthquakes such as Alaska 1964. This kind of displacement has occurred in the San Francisco Bay area in relatively recent geologic time, as shown by the structural origin of San Francisco Bay and the elevated marine terraces along the coast.

There is some evidence that suggests that at least parts of the bay may have undergone vertical displacement within the time man has occupied the shores of the bay. Louderback reports in *California Division of Mines and Geology, Bulletin 154*, page 87, that ten of the 425 shell mounds surveyed in 1908 under the auspices of the University of California extended below present sea level. He states that the Ellis Landing Mound in southern part of Richmond was built on a base of firm gravel, and at the time of the investigation, high tide rose 18 feet above the base. It is not clear, however, to what extent the relation of other mounds to sea level today are the result of tectonic uplift and subsidence or compaction of weak soil beneath a mound.

Though the possibility of regional vertical displacement cannot be ruled out, judging from the sense of movement within the San Andreas fault system over the recent geologic past, it is not very likely, and probably would not exceed 1 to 2 feet at the time of the next major movement.

As information becomes available from surveys now under way, we can expect to learn about faults in the elongate structural basin on San Francisco Bay that are not known today.[1] The density of faults within the basin is probably about the same as in the bordering uplands. Most of such faults, as is also true in the uplands, can be expected to be inactive, and therefore of no significant concern in siting man-made structures.

No faulting in historic time has been recognized within San Francisco Bay. However, active faults in the Hayward and Calaveras fault zones probably extend northward across San Pablo Bay, Carquinez Strait, and Suisun Bay.

Significance of the Historic Record

In making an evaluation of the historic record of damaging effects on man-made structures of the responses of different kinds of ground to seismic loading, two things must be kept in mind, (1) the design and construction of many of the structures would not meet standards today, and (2) many of today's structures have not faced the test of strong earthquake ground motion. Nevertheless, the record is clear in pointing up the kinds of problems involved in building on bay sediments for which engineering solutions must be sought. Furthermore, we can conclude that knowing how to design structures for reasonable safety to life and property is not enough. We must also foster development of public procedures such that our knowledge of how to cope with the problems of building on lands underlain by bay sediments is used.

The problems most difficult to cope with probably are those associated with ground failure:

1. Areas are now being developed for residential and commercial uses, where the top of the fill does not extend above high tide. Such areas are protected by perimeter dikes. The historic record demonstrates that steps must be taken to preclude failure, either of the dike itself or of the sediments underlying the dike. Failure of a dike at one place can result in flood damage to a relatively large area.

Tides and ground subsidence increase the problem in the southern part of San Francisco Bay. Here the mean tidal range is greater than in the north bay area. For example, it is 9 feet at Alviso and 5.9 feet at Point San Pablo. The maximum tidal range in the two places is 14 feet and 11 feet, respectively. Over the period 1934 to 1967, subsidence of the land due to ground water withdrawal increased from Dumbarton Bridge southward to about 6 feet at Alviso.

2. Use of San Francisco Bay as a harbor for commerce has long been, and will continue to be, the keystone of the economic structure of the bay area. Looking into the future, to accommodate larger ships now being built and planned will require deepening of the shipping channels from time to time. Present plans call for deepening the main shipping channels in the bay from 35 to 45 feet. Unless proper measures are taken, the potential for subaqueous slides under dynamic loading may increase as the channels are deepened, and concomitantly there could be an increase in the potential for serious hazard to some bayside locations either directly from slides and/or possibly from slide-induced waves. Also, subaqueous slides could cause serious damage to utility and communication lines. However, the majority of the channels to be deepened are not adjacent to areas currently planned for future development by BCDC.

3. The record shows that the phenomena of lurching, slumping, and ground cracking in weak, water-saturated sediments in the vicinity of free faces is common during strong earthquake loading. Therefore, slopes must be engineered to reduce this kind of hazard to an acceptable level. Furthermore, underground utilities in the vicinity of free faces must be engineered to withstand some cracking and movement of the ground toward the free face.

Seismic and Geologic Parameters

A list of criteria that define the natural environmental conditions that should be taken into account so that the best scientific information and technical judgment is utilized to avoid undue risk to life and property, must stem from our knowledge today. It must be recognized that little geologic data are available for large parts of the bay, its bordering marshlands and former marshlands. Also, there are large gaps in our understanding of the earthquake problem. Any list of seismic and geologic parameters should be revised from time to time as more data become available and our understanding and experience increase. It is understood also that any site should be evaluated from the standpoint of its area setting, and therefore it may be necessary to go beyond the boundary of a site to obtain needed information.

Seismic Parameters

1. The region is a seismic one, with a significant likelihood in a score or so of years of a potentially damaging earthquake or earthquakes nearby, most probably on the San Andreas, Hayward, or Calaveras faults.

2. Seismic velocities of bedrock and various types of bay sediments.

3. Ground response. It is well known that different kinds of ground respond differently to seismic loading. The relation between kind of ground and kind of response is not clearly known and understood. However, estimates of probable response can be made but should be used with caution for design studies. There are wide gaps in the record of the relation of ground motion to earthquake magnitude and depth of focus. The ground motion associated with a great earthquake (Magnitude 8+ comparable to the 1906 quake in the Bay area) has never been recorded instrumentally.

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[1] As used here, San Francisco Bay includes the bordering marshlands and former marshlands.

Geologic Parameters

1. Depth to bedrock and configuration of the bedrock bottom of the bay.
2. Nature, thickness, and distribution of older sequence of bay sediments.
3. Configuration of top of older sequence of bay ediments. This was an irregular land surface with a tream drainage pattern prior to inundation and depoition of the bay mud sequence within the last 10,000 o 20,000 years.
4. Nature, thickness, and distribution of younger equence of bay sediments (bay mud).
5. Presence or absence, thickness and extent, of iquefiable sand layers or lenses, particularly within the ay mud sequence. Special attention should be given o ascertaining the presence or absence of liquefiable and layers or lenses beneath locations for dikes and ear free faces.
6. Subsurface exploration (borings and geophysical tudies) should be carried to depths and horizontal xtent adequate to make a satisfactory evaluation of he risk in view of the use intended.
7. If a fill lies across the projection of a known ctive fault, or along the projection of a fault indiated by offset of recent sediments as revealed by subottom acoustical profiling, and if, in view of the se for which the site is planned, an underlying active ault could be a significant hazard, a survey should be ndertaken to determine the presence or absence of ne fault across the site being considered, and, based n expected activity, its potential effect on the site.
8. Land subsidence at the southern end of the bay being studied by the U.S. Geological Survey. It is ne result of ground water withdrawal. The extent rom 1934 to 1967 is shown on the map on p. 23 f this volume. The rate of subsidence has increased in ecent years. This subsidence can be controlled by inreasing the artesian head. Hopefully, this will be ccomplished some time in the future by decrease in round water withdrawal as water from the State's outh Bay Aqueduct and the San Felipe project beomes available over Santa Clara Valley.

SOIL AND FOUNDATION ENGINEERING CONSIDERATIONS

Questions to Be Considered Involving Geology, Soil, and Foundation Engineering

With regard to site stability and site behavior, an valuation will be required of all significant problems ncerning geologic hazards or soil behavior. The onsor should be prepared to describe appropriate eps which will be taken to restrict the effects of these oblems to acceptable limits. Specific problems to be nsidered would be:

atic Problems

1. Subsidence and differential settlement.
2. Landsliding and slope instability.

rthquake Problems

1. Surface faulting, ground rupture, and tectonic ovements.
2. Soil liquefaction.
3. Landsliding and slope instability.
4. Soil settlement including differential settlement.
5. Ground shaking.

The extent of investigation and analysis should be sufficient to define the potential severity of each particular type of problem, and the margin of safety provided in relation to any particular problem should be appropriately weighted in relation to the consequences of any failure or malfunction considered possible.

General Scope of Investigation

The investigation should include sufficient field study and literature review of the geology of the adjacent areas to establish a reasonable correlation between the conditions found on the site and those known to exist in the surrounding areas. Evidence should be given in the report to show how correlation was established. Methods used in the investigation should be accurately and adequately described.

Borings should be spaced close enough and should extend deep enough to adequately define the vertical and horizontal extent of the various soil deposits. These deposits in turn should be related to the areal geology, with the evidence for this relationship clearly stated.

An appendix should be prepared and either be included in the report, or be available upon request, giving a detailed description of the field exploration. This should include (among other items) descriptions of boring equipment and methods, sampling equipment and techniques, the degree of supervision and general background of the supervisory personnel, complete boring logs, detail of all field tests, and all other information that might be pertinent to establishing the thoroughness of the field investigation. An appendix also should be prepared and either be included in the report, or be available upon request, giving a detailed description of all laboratory testing. Testing equipment should be described in sufficient detail to satisfy a knowledgeable engineer of its adequacy (unless reference to some established standard will suffice). Test methods also should be described in similar detail. Special tests should be described and their relevancy to the project demonstrated.

The report should contain sufficient data and supporting information to allow any knowledgeable engineer to evaluate the results presented. In particular they should be described in enough detail to allow an evaluation both of the validity of the analyses and recommendations, and of the limits of confidence that might be applicable to them. Calculations should either be included as an appendix, or should be available upon request.

The scope of possible investigations relative to "Static Problems" and "Earthquake Problems" is discussed in the succeeding paragraphs. The discussion is not intended as a specification which must be rigidly followed but rather as an indication of the depth to which the problems should be investigated to prepare a satisfactory report for a proposed development or project.

Static Problems

Site Conditions

In order to properly evaluate a proposed site with regard to problems which might be encountered in placing fills on soft bay mud or similar materials, it is necessary to make a thorough investigation of the history of the site. The investigation should be such as to determine the presence or absence of visible or obliterated sloughs and ditches, levees and previously filled areas; length of time area has been free of tidal action; whether or not the area has been cultivated and for how long; the degree of desiccation; thickness and nature of the so-called "crust" and other factors which would influence the behavior of the fill and improvements constructed thereon.

Subsidence and Differential Settlement

The possible effects of long-term subsidence (due to lowering of the ground water) must be considered in the southern end of the bay (see "Geological and Seismological Considerations," p. 122). The final elevation of fills should be adjusted for such subsidence.

Settlement studies for fills on bay mud and other soft foundation materials must give consideration to: consolidation of the underlying foundation materials; settlement within the fill proper; plastic flow of the supporting foundation materials; variations in the thickness of mud throughout the filled area; lateral movement of the mud resting on sloping surfaces of firm material or bedrock; and the rate of settlement. Adequate studies are required to determine the expected differential settlement within the area being filled and differential settlement to be anticipated within specific building sites. Analyses may also be required to determine whether differential settlement will occur between the peripheral portions of the area being filled and the surrounding areas and the effects of such differential settlement. For pile supported structures the downdrag created by placement of fills on compressible soils such as bay muds must be thoroughly investigated to determine the effect on proposed structures.

The possible effects of adjacent construction, either within the site being investigated or future reasonably possible construction immediately outside the site, should be analyzed in sufficient detail to ascertain that adverse conditions will not develop from such construction. The possible variations in water table elevation in the fill as well as boundary water levels require sufficient analyses to determine their influence on the performance of the fill relative to settlement and other factors such as bearing capacity and stability. It is desirable that records of the rate of settlement due to soil consolidation be obtained for bay fills, and made available to the public as an aid to ascertaining the satisfactory performance of the fill.

Bearing Capacity

It is necessary to determine the ability of the fill to support the loads which will be imposed thereon with appropriate consideration being given to: the strength of the fill material and the underlying foundation materials; type and depth of foundations to be utilized; thickness of the fill material required to reduce foundation loads to acceptable proportions; the type of loading anticipated; and other pertinent factors.

Stability

Sufficient studies to demonstrate the stability of the slopes of the fill, both exterior and interior (for canals, lagoons and ponding areas), are considered essential. These should include consideration of such factors as strength of fill and foundation materials, variations in water surfaces, variations in moisture content and height of water table in the fill and the reasonable possibilities of future development of dredged channels adjacent to the fill. In addition to analyzing the stability of slopes, particularly exterior ones, the need for protection of the slopes from wave action requires an investigation to determine the type of protection required, and the extent, depth, and height of the protection relative to the adjacent water surface.

Construction Considerations

Since the performance of fills placed on soft foundation materials is related to the methods used in constructing the fill, it is necessary to present a detailed discourse on the proposed construction procedures. Such a discussion should include but not be limited to: a description of the proposed treatment of the surface to receive the fill including special provisions to be made for existing or obliterated waterways, levees and existing fills; type of fill material to be utilized and proposed methods of placement; special measures proposed to control within tolerable limits plastic flow of the foundation material and/or lateral movements on sloping surfaces.

Earthquake Problems

Surface Faulting, Ground Rupture (Lurching, Etc.), and Tectonic Movements

Reference information or results of explorations should be adequate to define the locations of significant fault zones on the site or close enough to the site to materially influence the site stability. In the event that geologic fault structures are found to exist within or near the site, an evaluation should be made of the probability of future activity on these faults in relation to other known fault zones in the San Francisco Bay Area. In the event that fault movement is indicated as being of such a probability as to require consideration in the design and development of facilities, an estimate shall be made, with appropriate supporting data, as to the extent of movement which should be anticipated for design purposes and the estimated probability of occurrence. In the event studies indicate serious possibility of tectonic movements involving uplift or subsidence of large land areas, recommendations shall be made concerning the magnitude and location of such ground surface movements which should be considered in the design of facilities.

Soil Liquefaction

In all projects where loose or medium dense saturated silt or sand strata exist, whether within the fil in the foundation material, or in backfill behind waterfront bulkheads, a careful appraisal should be made f the possibility of liquefaction of the soil during arthquakes and the possible consequences of such quefaction. For this purpose, previous experience vith respect to soil liquefaction during earthquakes hould be utilized to the fullest possible extent together vith appropriate analytical techniques. It should be ecognized that broad generalizations cannot be made n connection with liquefaction phenomena, and each ase should be evaluated separately with due consid-ration being given to the maximum probable severity f earthquake ground motion, the depth of the water able, and the density and strength properties of the articular sand or silt deposit. In cases where liquefac-ion may develop, steps should be taken such as the se of greater depth of soil above water table, densi-cation of the soil, or other measures to alleviate the ossibility of adverse behavior.

andsliding and Slope Instability

A complete evaluation should be made of the possi-ility of landslides occurring due to:

1. Slides due to liquefaction (in saturated cohesion-ess soils, or sand seams or lenses in clay deposits),
2. The driving forces exceeding the resistance of aturated fill, or foundation soils, or
3. Surface slumping of cohesive fill slopes.

All slopes should be conservatively designed to re-uce the risk and consequences of landslides to accept-ble limits.

Particular attention should be given to evaluate the verall site stability to assure that the site can be ex-ected to remain substantially intact during and sub-equent to the maximum credible earthquake. Al-hough some cracking of the ground and cracking of avements might occur, it should be expected that here would be no large fissures, offsets, or lateral ovements or vertical slide movements of more than few inches.

The evaluations of possible landslides should include onsideration of both perimeter and interior slopes, d the investigation should be planned to adequately aluate the probable presence or absence of soils hich might experience severe loss of strength or quefaction conditions during an earthquake.

In evaluating landslide potentials, as well as locations soil strata which might be subject to reduction of rength or liquefaction potential, it is recognized that omplete certainty in the locations of such strata and the evaluation of behavior during an earthquake is t practically feasible. Therefore, the studies should carried to a degree of thoroughness which would dicate a high order of dependability of the overall onclusions. The recommendations reached should in-ude an appropriate evaluation of the limits of confi-ence which might be expected based on the extent studies made.

bsidence and Differential Settlement

In addition to considering problems due to subsid-ce, and determining that the settlement within the l itself will be within permissible limits, an evaluation so should be made of the settlements likely to de-lop during an earthquake due to the compaction of underlying foundation soils. Such settlements may vary from negligible amounts for clays to substantial amounts for loose cohesionless soils. These evaluations should include estimates of both the expected general areal settlement and the degree of non-uniform settlement which might be expected under the effects of earthquake shaking.

Intensity, Duration, and Character of Ground Shaking

After ensuring that all major stability problems involving permanent displacements of fill or foundation materials have been eliminated or reduced to tolerable proportions, consideration should be given to the probable nature of the ground shaking to which structures will be subjected. These motions may not be influenced significantly by foundation type (footings, piles, piers, etc.), but they will depend in large measure on the magnitude of the earthquake, the epicentral distance, and the nature of the entire soil deposit (including any fill (overlying base rock at the site. From a knowledge of the soil conditions and the probable magnitude and location of the design earthquake, it is possible to estimate a response spectrum characterizing the nature of the ground surface motions and their influence on structures. This is particularly necessary in the case of fills over deep layers of bay mud, since the response spectrum for these conditions is likely to differ appreciably from the average spectrum often used for civil engineering design purposes. Buildings and other facilities at the site should then be designed in accordance with the changed response spectrum indicated at the site, in addition to the static stresses imposed by normal loads and the effects of any anticipated differential settlements. In general, since the predominant period of the ground motions is likely to be relatively long for areas underlain by thick layers of bay mud, seismic forces due to shaking are likely to be least for short period one-story structures of moderate floor area and will be relatively larger for multi-story structures.

Design of Structures and Facilities

Foundations

Since the soil underlying a building will be subjected to some degree of horizontal and vertical displacements during an earthquake, it is desirable to minimize differential displacements at the base of the structure as far as possible.

With regard to the performance of pile foundations during an earthquake, there is good evidence to show that the use of piles in areas subjected to earthquake-induced settlements often can prevent major damage to the buildings. However, in the design of pile foundations, the following precautions should be observed:

1. Presently available evidence indicates that the presence of piles is not likely to affect significantly the nature of the ground shaking to which a building is subjected.
2. If piles are supported in granular soils in which settlement may be induced by the ground shaking, settlement and possibly differential settlement of the building foundation will result.

3. If the soil around a pile settles during an earthquake, the pile will be subjected to an additional downward force by the surrounding soil.

4. Even if the soil at the lower ends of the piles provides adequate supporting capacity, liquefaction of a substantial section of loose cohesionless soils above the pile tips may permit lateral movement of the pile caps accompanied by bending of the piles.

5. Vertical piles are not likely to provide any substantial degree of resistance to translation of a building due to lateral slippage in the underlying soils.

Retaining Walls and Basement Walls

Retaining walls and basement walls should be designed to withstand the seismic forces developed in the backfill materials or the forces which might be produced as a result of the interaction of the structure and the foundation soils in addition to the usual static lateral pressures. It is recognized that basement wall design involves substantial judgement based on experience since performance has been better than present theory might suggest.

Utilities

Special precautions should be taken as appropriate in the desgn and construction of utility systems to provide sufficient flexibility for the systems to withstand the expected ground movements during an earthquake. Additional precautions may be advisable in some cases, such as automatic shutoff valves, to minimize hazards in the event the system is disrupted, and independent supplemental water systems (or equivalent) may be called for.

Conclusions

In view of the many different uses which may be planned for reclaimed lands areas and the variations in the geologic, soil, and foundation problems which require consideration, it is not feasible to prescribe specific investigations, tests, or analyses which would be appropriate for all of these varied requirements. The objective of the foregoing discussion, therefore, has been directed toward outlining the nature of the problems which might require consideration in any specific project. It is believed that the implementation of policies required to answer these problems will require the formulation of a highly competent review board which will be charged with the responsibility of evaluating the appropriateness of the specific investigations and analyses which may be required for any particular project. The appropriate scope and extent of such investigations and studies should be sufficient to enable the knowledgeable professional engineers, geologists, and other specialists on the Board of Review to ascertain that the severity of each particular type of problem has been reasonably evaluated, and the margins of safety provided are appropriate in relation to the consequences of occurrence of the particular problem under consideration. It is expected that, as work is carried on under this program, there will develop a sound body of information concerning investigative and design and construction procedures which will enable desirable projects to be carried out with an optimum balance between the factors of cost, risk, and function, and that this can be accomplished while encouraging a continued improvement in the "state of the art" regarding application of technical knowledge to advantageous use of the properties concerned.

ENGINEERING SAFETY REQUIREMENTS

The following is presented in qualitative, not quantitative, form to give a general summary indication of the type and scope of the occupancy, the required scope of investigation, and construction requirements for each risk zone. The details and lines of demarcation should have an intensive review and further study by a highly qualified group such as the proposed future Fill Review Board.

The four risk zones will first be defined, based on various foundation and geological conditions. Typical occupancies and structure types to be expected in each zone are outlined, together with the foundation and geological investigation required for these typical uses (Table 1). It is usually possible in each zone under *special conditions* to construct a building with either normally higher risk occupancies or of normally higher risk types of construction, but then additional investigation is required and special construction precautions must be observed (Table 2).

Risk Zones

Zone I—Minimum Risk

This zone would comprise those areas or situations where:

a. There is a minimum depth of recent bay muds underlying the proposed fill so that the fill is not subject to significant settlements and/or differential settlements under either static or seismic loads.

b. The underlying material is not subject to liquefaction under seismic loads.

c. The proposed fill and its underlying material are not subject to significant slumping, and/or lurching and/or slides under either static or dynamic loads.

d. The area is not in an active fault zone or in extensions of active fault zones.

Zone II—Moderate Risk

This zone would comprise those areas or situation where:

a. There is such a depth of recent bay muds underlying the proposed fill, that substantial settlement and/or significant differential settlements may be expected under either static or seismic loads.

b. The underlying material is not subject to liquefaction under seismic loads.

c. The fill and its underlying material are not subject to significant slumping, and/or lurching, and/or slides.

d. The area is not in an active fault zone or in extensions of active fault zones.

Zone III—Substantial Risk

This zone would comprise those areas or situation where:

a. There is such a depth of recent bay muds underlying the proposed fill, that substantial settlement and r significant differential settlements may be expected nder either static or dynamic loads.

b. The underlying material is subject to liquefaction only local areas adjacent to sloughs and other feaures with free faces.

c. Small limited zones may slump, and/or lurch, nd/or slide.

d. The area is not in an active fault zone, or in exensions of fault zones.

one IV—Maximum Risk

This zone includes those areas or situations where:

a. There are deep compressible bay muds where ajor settlements or differential settlements are probble, unless suitable precautions are taken to bring the ettlements within acceptable limits under static or eismic loads.

b. The areas are adjacent to channels where abrupt hanges in the topography are combined with strucurally questionable foundation materials.

c. Formerly uncontrolled dump areas exist.

d. The area is within known active fault zones or extensions of active fault zones.

Normal Condition Categories of Occupancies, Investigations, and Construction

ategory 1

ccupancies O-1

(Normally reserved for Zone I, or alternatively one II under Special Conditions)

1. Occupancies and structures in this category ould not be specially limited for safety reasons.

2. Occupancies and structures suitable for construcon elsewhere in California.

vestigation Requirements I-1

1. Sufficient investigation to determine that the area oes not fall into a higher risk zone.

2. Adequate investigation to suit the type and imortance of the structures. Sufficient soil investigation determine the anticipated amount of settlements and ifferential settlements. Settlement correction studies ould be required depending on the amount of these ticipated settlements.

3. Ground response studies and building response udies shall be made for all structures over ......[1] ories, or having roofed-over gross total floor areas reater than ......[2] sq. ft., or having occupancy loads reater than ......[2] persons.

4. Records of instruments, borings, surveys, settleents, and all data submitted are to be public records, ailable to all.

onstruction Requirements C-1

1. There would be no difference from minimum uilding construction elsewhere, except that all underround utilities must be given special earthquake conderation. Water and gas systems should be specially alved to minimize the effects of possible pipe breakge during earthquakes. In developments over..........[3] acres, earthquake strong motion measuring instruments are to be required. Adequate freeboard and structurally stable dikes must be provided for structures adjacent to Bay waters if surface of fill is not above highest tide level.

2. All buildings four or more stories in height (or approximately equivalent for other structures) shall have a structural engineer[4] of record who, upon completion of the construction and before occupancy, will file a statement with the local building department that he has knowledge that the structural engineering aspects of the plans and specifications have been complied with during construction.

3. All buildings over 10 stories (or approximate equivalent for other structures) shall have earthquake strong motion recording equipment.

4. Where strong motion instruments are required, the owner shall furnish such space as required to house the instruments and provide access to such space for service and reading. Records from strong motion instruments are to be public records, available to all. The owner shall furnish and maintain, or pay sufficient fees to furnish and maintain ......[3] instruments per ......[3] sq. ft. of building or per ......[3] acres whichever is intended elsewhere in these requirements. (As alternate to the foregoing, possibly more useful data would be obtained if the future Board of Consultants selected sites for instruments, based on geologic and soil conditions. Cost of installation would be borne by fees paid by each developer whether or not instruments were placed on his land or in his buildings.)

Category 2

Occupancies O-2a and O-2b

(Normally reserved for Zones I & II, or (alternatively to Zone III under Special Conditions.)

O-2a Occupancies (Typical Minimum Occupancies and Structures):

1. Single family and multiple family occupancies of not more than two story wood frame construction.

2. Other occupancies shall be no higher than one story plus mezzanine.

O-2b Occupancies:

1. Adequately designed industrial plants, refineries, airport and harbor structures, and similar occupancies.

Investigation Requirements I-2

1. Sufficient investigation to determine that the area does not fall in a higher risk zone.

2. For dwellings, sufficient investigation on an area (not lot) basis to determine the anticipated amount of differential settlements to be used for design.

3. For non-dwellings, sufficient soil investigation to determine the anticipated amount of settlements and differential settlements, to make settlement correction studies if applicable, and other special design criteria if applicable.

To give a concept of the number of stories—"10" is suggested. Industrial plants, refineries, airport and harbor structures, etc., to be evaluated individually.

To be determined by the successor board.

To be determined by the successor board.

[4] The question whether this responsibility can be legally assigned to only a structural engineer (rather than also to an architect, civil engineer, or other licensed professional) has not been resolved by legal counsel at this writing. The non-legal aspects also remain unresolved due to time limitations. For convenience the term "Structural Engineer" is used here and in following sections within the limitations stated herein.

Construction Requirements C-2

1. All construction requirements of Category C-1.
2. A structural engineer of record is required for all buildings having structural elements of concrete, brick, hollow concrete block, or other unit masonry. A structural engineer of record is also required for all industrial buildings, refinery structures, and airport and harbor structures. Residential masonry fireplaces properly reinforced and veneer properly anchored are considered nonstructural.
3. An independent supplemental water or equivalent system for fire protection.

Category 3

Occupancies O-3

(Normally reserved for Zones I, II, III, or to Zone IV under Special Conditions.)

Typical Minimum Occupancies and Structures: Occupancies would be limited to water-oriented industries, wharves and docks, warehouses, storage areas, and similar minimum population density occupancies. Offices shall be only those necessary to accommodate these occupancies. Structures shall be one story in height, unless of very flexible construction. Only public buildings which are necessary to service the area are allowed. Fire stations could be placed in this zone if mandatory from other considerations.

Investigation Requirements I-3

1. Sufficient investigation to determine that the area does not fall in Zone IV. Includes all requirements for Categories I-1 and I-2.
2. Foundation investigation to determine extent and limits of possible liquefaction, or slides.
3. Individual site investigation to determine the anticipated amount of settlement and differential settlement, and to make settlement correction studies if applicable.

Construction Requirements C-3

1. All requirements of Categories C-1 and C-2.
2. Structural engineer of record for all buildings.
3. Buildings located to avoid probable defectiv foundation areas.
4. Special protection requirements for all underground utilities in addition to those in Category C-1
5. Tied footings, beam interconnected pile caps or mat foundations.
6. Structural system compatible with anticipated settlements and/or horizontal movements.

Category 4

Occupancies O-4

(Normally reserved for Zone IV). Typical Minimum Occupancies and Structures:

1. Parks, golf courses, and similar land use.
2. Appurtenant structures (such as storage sheds) if of one story wood frame construction or equivalent light mass, inherent strength, and ductility.

Investigation Requirements I-4

There must be sufficient investigation that these areas are safe and suitable for use under *normal* conditions: ground failures may occur during earthquake conditions.

Construction Requirements C-4

1. Only light, small wood or steel structures.
2. Telephone and electric power to be above grade Special precautions for water and power. Gas distribution systems shall be limited or prohibited where possible.

Table 1. Allowable occupancies, required investigations and construction requirements—normal conditions.

Zone	Occupancies	Investigations	Construction Requirements
I	O-1	I-1	C-1
	O-2a & b	I-1	C-1
	O-3	I-1	C-1
	O-4	I-4	C-4*
II	O-2a & b	I-2	C-2
	O-3	I-2	C-2
	O-4	I-4	C-4*
III	O-3	I-3	C-3
	O-4	I-4	C-4*
IV	O-4	I-4	C-4

* Applicable provisions only.

Occupancies	Description
O-1	All occupancies.
O-2a	Single family and multiple family occupancies not more than two story wood frame construction. Other occupancies no higher than one story plus mezzanine.
O-2b	Adequately designed industrial plants, refineries, airport and harbor structures, etc.
O-3	Water oriented industries, wharves and docks, warehouses, storage areas, and similar minimum population density occupancies. (See Category O-3 for more detailed description)
O-4	Parks, golf courses, and similar land use.

Special Conditions

If, for economic or other reasons, it is desirable to nstruct a facility with higher density of occupancy more basically vulnerable type of construction in nigher risk zone, this can be done (with certain exptions) by making a more extensive investigation d providing additional construction precautions as dicated by such investigations in addition to those ted below. However, under circumstances deemed cessary by the Fill Review Board, the Fill Review ard may ask for more data, information, and investition than expressed below.

ecial Condition 1

Since Zone I is the minimum risk zone, no *extra* ecautions need be taken.

ecial Condition 2

In order to use occupancies or construction norlly reserved for Zone I in Zone II, the following *ditional* investigations and construction requirements mandatory in addition to the requirements of Catery 1 and Category 2.

ditional Investigation I-2A

Investigation Required for Higher Use Occupancies:

1. Detailed foundation investigation for individual es.

2. Foundation investigation to determine the anticied amount of differential settlements, and to make tlement correction studies if applicable.

. Ground response studies and building response dies shall be made for structures over ..... [5] stories height, or having roofed-over gross total floor areas ater than ..... [5] sq. ft., or having occupancy loads ater than ..... [5] persons.

nstruction Requirements C-2A

Construction Required for Higher Use Occupancies:

. All buildings over five stories to have ductile ment resisting frame. Rigid structures will have cial design requirements.

. Code requirements as to *either* strength *or* safe ral deformations to be increased in proportion to und amplification.

. All structures over 5 stories in height (or equiva-) to have earthquake strong motion recording ipment.

. Tracts over ..... [5] acres shall have an earthquake ng motion instrument which is independently sed and located away from buildings. There shall one such instrument for each ..... [5] acres.

Special Condition 3

In order to use occupancies or construction normally reserved for Zone II in Zone III, the following *additional* investigations and construction requirements are mandatory, in addition to the requirements of Category 3.

This would only be permitted if suitable foundation corrective measures are taken to reduce to a tolerable degree the effects of foundation liquefaction, sloughing or settlement.

Additional Investigation I-3A

1. Same as I-2A.

Construction Requirements C-3A

Construction Requirements for Higher Use Occupancies:

1. All buildings to have ductile moment resisting frame. Rigid structures will have special design requirements.

2. All structures over two stories in height or having roofed-over total gross floor areas greater than ..... [6] sq. ft. to have earthquake strong motion recording equipment.

3. Public assembly occupancies greater than 1,000 not permitted in roofed-over areas.

Special Condition 4

In order to use occupancies or construction normally reserved for Zone III in Zone IV, the following additional investigations and construction requirements and limitations are mandatory, in addition to the requirements of Category 3.

This would only be permitted if suitable foundation corrective measures are taken to reduce the effects of foundation liquefaction, sloughing, or settlement, and the effects of abrupt changes in topography or associated problems.

Additional Investigation I-4A

1. Same as I-2A.

Construction Requirements C-4A

1. There shall be no occupancies in Category O-3 located in active fault zones.

2. Structural engineer of record is required for all construction.

3. Earthquake strong motion recording equipment must be installed in all buildings.

4. Buildings shall not exceed two stories in height.

5. Earthquake design requirements to be increased or modified as required by the Fill Review Board.

6. Developments, including buildings therein, must have level survey-nets tied into permanent and established bench marks.

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be determined by the successor board.

[6] To be determined by the successor Board.

*Table 2. Allowable occupancies, required investigations and construction requirements—special conditions.**

Zone	Occupancies	Investigations	Construction Requirements
I	....	....	....
II	O-1	I-2A	C-2A
III	O-2a & b	I-3A	C-3A
IV	O-3	I-4A	C-4A

* More extensive investigations and more restrictive construction requirements *in addition* to those in Table 1. See requirements under Special Conditions 2, 3, and 4 for other limitations and additional requirements not noted in this table.

FUNCTIONS AND COMPOSITION OF PROPOSED CONSULTANT BOARD

It is recommended that the San Francisco Bay Conservation and Development Commission, as a part of its plan for the bay, include in their plan a Consultant Board (Fill Review Board).

Functions

1. To establish, and to revise, safety criteria for bay fills and structures thereon with respect to risk zones.

2. To review all except minor projects for the adequacy of their specific safety criteria, and to make recommendations concerning these criteria.

3. To gather, and to make available, data developed from specific projects under their jurisdiction.

4. To complement the functions of local building departments and local city planning departments.

Composition

1. To have eleven (11) members. One member chosen at large and 10 members, each of whom is a recognized authority in one of the following disciplines:

Geology (3 members)
Soils engineering (3 members)
Structural engineering (3 members)
Architecture (1 member)

2. Of the eleven (11) members, to have no more than five (5) members whose principal employment lies in one of these fields:

Private employment
Academic employment
Governmental employment

3. Members are to be selected by the successor organization to the San Francisco Bay Conservation and Development Commission.

4. Members are to have three (3) year terms of office, with the terms of office to be staggered.

5. To be paid expenses plus a nominal sum for attending board meetings, consistent with conventional practice for these types of boards.

6. There should be adequate funding to provide sufficient staff support for the board and sufficient staff to implement the policies of the board.

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